10-28-2013

Laboratory and Field Evaluations of Pervious Concrete

Mandar Dewoolkar
University of Vermont, Mandar.Dewoolkar@uvm.edu

Ian Anderson
University of Vermont

Mark Suozzo
University of Vermont

A Report from the University of Vermont Transportation Research Center

Laboratory & Field Evaluations of Pervious Concrete.

TRC Report 13-007 | Anderson, Suozzo, Dewoolkar | October 2013

UVM TRC Report #13-007

Laboratory and Field Evaluations of Pervious Concrete

October 28, 2013

Prepared by:
Ian A. Anderson, Graduate Student, Civil and Environmental Eng.
Mark Suozzo, Graduate Student, Civil and Environmental Eng.
Mandar M. Dewoolkar, Associate Professor, Civil and Environmental Eng.

College of Engineering and Mathematical Sciences
Votey Hall
33 Colchester Ave.
Burlington, VT 05405

Acknowledgements

This work was funded by the United States Department of Transportation through the Transportation Research Center (TRC) at the University of Vermont (UVM) along with the Vermont Agency of Transportation (VTrans). These two sources of research funding are gratefully acknowledged. The authors are grateful to the VTrans Materials & Research Section for their assistance in sample preparation. Additional thanks go to Bradford Berry, George McCain, and Lalita Oka who provided assistance on this project.

ABSTRACT

This study evaluates the factors affecting the testing of strength and hydraulic parameters of pervious concrete pavement (PCP), presents results of long-term infiltration monitoring and cleaning operations, and investigates freeze-thaw durability of pervious concrete and the effects of fly ash. The specific objectives of this study were to: (1) determine how rubber capping and sulfur capping affect compressive strength measurements of PCP; (2) determine the effects of height to diameter (H:D) ratio of cylindrical specimens on compressive strength measurements of PCP; (3) compare various methods used to determine infiltration rate of PCP in the field to one another and to laboratory measurements of hydraulic conductivity; (4) monitor two PCP facilities in Vermont for changes to infiltration rate over time; (5) evaluate the effects of various cleaning methods on the restoration of infiltration rates; (6) determine the effects of deicing salts on pervious concrete, using a modified and more field representative testing procedure that involves slow freeze-thaw cycling in drained condition, and (7) determine the effects of cement replacement with increasing amounts of fly ash on the freeze-thaw durability of pervious concrete. In addition, the results of this study were used to suggest correlations to the field observations seen at several pervious concrete sites in Vermont.

Capping with rubber pads was found to provide more consistent compressive strength measurements compared to sulfur capping for both H:D ratios studied. H:D ratios less than the standard 2:1 were found to increase results of compressive strength measurements; however, a ratio of 1:1 was found to provide inconsistent results. Compressive strength specimens with H:D ratios less than 2:1 can be divided by 1.1 to estimate compressive strength of 2:1 H:D specimens. Results of laboratory hydraulic conductivity, single ring, double ring and falling head infiltrometer testing were found to correlate linearly to one another with a relation of 1.0 : 1.8 : 1.5 : 9.0 for 6” thick PCP. Long-term field monitoring of infiltration rates indicated reductions of 59% and 26% for the facilities investigated. Cleaning methods were found to differ in effectiveness, with average restoration rates of 21% for street sweeping, 30% for vacuum truck cleaning, 85% for pressure washing, 10% for hand vacuuming, and 100% for combined pressure washing and vacuuming. Sodium Chloride deicing salt at 8%, followed by 4 and 2% resulted in the greatest freeze-thaw damage. Water alone did not result in damage during 100 one per day freeze-thaw cycles. Fly Ash replacement of 10 and 20% showed a decrease in freeze-thaw damage as compared to a control mix with no fly ash replacement.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT AND RESEARCH OBJECTIVES

Over the past century urban centers have been growing with new buildings, roads and parking lots increasing the amount of impervious surface in given watersheds. This trend is expected to continue and has well-established impacts on the hydrological cycle, water quantity and water quality of an area (Sansalone, 2008). Quantity of runoff can increase greatly when an area is converted from native cover to impervious surfaces. Impervious surfaces collect pollutants such as sediments, heavy metals, nutrients (including phosphorus and nitrogen), oils, grease and fecal pathogens; these pollutants are then transported into water bodies during storm events. Impervious surfaces also reduce the amount of water infiltrating into the underlying soils, affecting local groundwater levels. Due to these negative impacts, federal, state and local governments have implemented stormwater control measures that require runoff from impervious surfaces be treated with the goal of increasing water quality and to ensure that the local hydrology is not negatively impacted (Tennis, 2004). The Clean Water Act of 1999 established regulations on stormwater runoff and set water quality standards for stormwater runoff.

To meet these regulations various stormwater control measures have been used. Pervious pavement systems have been developed as an innovative solution to handling stormwater while providing for structural needs of pavement materials. Where impervious surfaces can collect pollutants and then wash them away during storm events, pervious pavements allow water to infiltrate into the surface and be treated in the subsoil (Tennis, 2004). In addition to having environmental benefits, pervious pavements have several secondary advantages when compared to traditional pavement materials. Pervious pavements reduce noise, provide increased skid resistance, diminish the heat island effect and remove the need for additional stormwater control structures (Ferguson, 2005). Pervious pavement has several disadvantages as well; they are typically weaker compared to traditional pavements, periodic maintenance is required to keep adequate permeability, and their performance in cold climates is not well understood compared to traditional pavements. This research focuses on pervious concrete pavements (PCP) which incorporate a uniformly graded aggregate (typically 3/8" in diameter), little or no sand, water and chemical admixtures.

Several laboratory studies have been conducted to increase the viability of pervious concrete in northern climates by increasing strength, improving resistance to freeze-thaw cycles and preserving long-term permeability (Schafer et al., 2006; Chopra et al., 2010). However, differences exist between results of field testing and laboratory studies (Henderson et al., 2009). In order to expand the applicability of pervious concrete a better understanding of how field and laboratory test methods compare to one another is needed. This research focuses on evaluating the methods used to determine the hydraulic and mechanical properties of pervious concrete. Results allow for better understanding of various testing methods and how they compare to one another. In addition to comparisons of testing methods mix designs incorporating fly ash are evaluated to determine if the freeze-thaw durability can be increased for a field representative test, with salt exposure. Finally, maintenance practices for pervious concrete pavements such as street sweeping, vacuum truck cleaning, pressure washing, and hand vacuuming are evaluated to determine how these practices affect the infiltration rate of pervious concrete pavements in northern environments.

The specific objectives of the study presented here are to:

- determine the effects of rubber capping and sulfur capping on compressive strength measurement of pervious concrete;
- determine the effects of height to diameter ratio on compressive strength measurement of pervious concrete;
- correlate results of single ring, double ring and falling head methods used to determine infiltration rate in the field;
- correlate results of field infiltration methods to saturated hydraulic conductivity measurements;
- monitor two field sites for infiltration rate and damage due to winter maintenance over a year-long period;
- evaluate the effectiveness of cleaning methods for pervious concrete sites;
- determine the effects of deicing salts on pervious concrete, using a modified and more field representative testing procedure that involves slow freeze-thaw cycling in drained condition; and
- assess the effects of cement replacement with increasing amounts of fly ash on the freeze-thaw durability of pervious concrete.

Chapter two presents literature review of previous studies conducted on pervious concrete. Three manuscripts are presented in the subsequent chapters. Chapter three examines the effects of various testing parameters on compressive strength and compares infiltration measurement methods to hydraulic conductivity measurement methods. This manuscript is under consideration with the *ACI Materials Journal*. The fourth chapter presents results of long-term field monitoring and evaluation of various cleaning methods. This manuscript is in print with the *Transportation Research Record,* Journal of the Transportation Research Board. Chapter five examines the effects of fly ash and the exposure to salt on the freeze-thaw durability of pervious concrete. This manuscript is being submitted to the *ACI Materials Journal*. Each chapter was written to be a complete article and contains background information, methods, results and references. A summary of the overall conclusions and recommendations for future work are presented in chapter six.

CHAPTER 2

LITERATURE REVIEW

2.1 BACKGROUND

Literature related to pervious concrete pavements, their role in stormwater control and additional benefits are presented along with a description of materials commonly used to make pervious concrete. Engineering properties such as strength, permeability, and void ratio, as well as the freeze-thaw resistance and test methods used to determine these properties are presented in this section.

2.2 PERVIOUS CONCRETE PAVEMENTS

PCP is defined by ACI (2010) as a concrete mix design that consists of a uniform coarse aggregate (3/8” in size is most common), cement, water, and can include admixtures and/or supplementary cementitious material. Pervious concrete pavements (PCP) differ from traditional concrete pavement systems due to the lack of fines and use of uniformly graded aggregate creating large interconnected voids (Ferguson, 2005). These voids typically comprise 25%-30% of the total volume of pervious concrete; allowing for connections between the top and bottom of the pavement surface. A thin coat of cement paste surrounds the aggregate providing rigidity and strength (Ghafoori and Dutta, 1995a). Pervious concrete has been used in several ways including (1) concrete walls where lightweight construction is required, (2) base course for underlying city streets, (3) bridge embankments, (4) beach structures and seawalls, and (5) surface course for parking lots, low-volume roads and driveways (Ghafoori and Dutta, 1995b). For the purposes of this study pervious concrete will be investigated for use as a surface course paving material.

2.3 ADMIXTURES

Aside from coarse aggregate, cement and water, pervious concrete can also incorporate high-range water reducers, air entraining agents, viscosity modifying admixtures, fly ash and silica fume (ACI, 2010). High range water reducers are added to decrease the water demand of the concrete, resulting in higher compressive strength values. Air entraining admixtures are added to improve freeze-thaw resistance of traditional concrete and have been adapted for use in pervious concrete. The low workability of pervious concrete can be improved by adding viscosity modifying admixtures to increase the flow of the cement paste surrounding the aggregate resulting in better compaction.

2.4 FLY ASH

Fly ash is a byproduct of the combustion of coal used for generation of electricity. Of the fly ash generated through coal energy combustion, 20% has been used annually in concrete production (Helmuth, 1987). As a byproduct material, there is no associated carbon dioxide produced to use fly ash, and if unused fly ash is discarded into landfills. Cement production by comparison is responsible for 5% of global carbon dioxide production (Worrell et al., 2001). Fly ash is a small spherical particle, typically 0.2-10 μm, which occurs when mineral impurities fuse during combustion (Chindaprasirt et al., 2005). By comparison, the particle size of type I-II cement on average is between 10-20 μm (Bentz et al., 2008). Fly ash is categorized based on ASTM 618 (2010), *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*, with most concretes incorporating class C or F fly ash. Fly ash is the most commonly added supplementary cementitious material, with about 50% of all ready-mix concrete incorporating some amount of fly ash (PCA, 2002). Fly ash addition in conventional concrete has been shown to reduce water demand, similar to chemical water reducing admixtures (Helmuth, 1987). The small spherical particles act to lubricate the cement, improving workability, and can extend the set time (Chindaprasirt et al., 2004). Fly ash has been shown to increase the long term compressive strength of conventional concrete, but requires curing beyond the typical 28 days (Chindaprasirt et al., 2004; PCA, 2002). The smaller particle size relative to cement allows for a greater distribution of particle sizes, which can act to reduce the pore sizes of the cement paste (Chindaprasirt et al., 2005). Finer grained fly ashes were shown to further reduce porosity, pore size, and improve strength and workability (Chindaprasirt et al., 2004; Chindaprasirt et al., 2005). While fly ash has been used in conventional concrete for some time, little is known about its possible effects when incorporated into pervious concrete.

2.5 STORMWATER CONTROL

Traditional stormwater control systems, such as retention ponds or constructed wetlands, operate by collecting runoff from impervious areas and storing the water where it will be infiltrated or slowly discharged into a nearby water body. These systems require large amounts of land and continued maintenance to ensure the long-term performance (Ghafoori and Dutta, 1995a). PCP eliminates the need for additional stormwater control structures by allowing water to pass through the pavement material, reducing or eliminating runoff. During a storm event water infiltrates through the PCP and is stored in a gravel subbase; where the water can infiltrate into the native soils or be piped away for further treatment. Backstrom (2000) showed that groundwater levels under pervious pavement were significantly higher than levels under impermeable pavements, indicating that water infiltrated through pervious pavements to the native soils to provide groundwater recharge.

Investigations into the water treatment potential for pervious pavements have shown that these systems can effectively treat stormwater through a combination of mechanical and biological methods. Pratt et al. (1996) and Schueler (1987) demonstrated that pervious pavement systems, including pervious concrete, remove large amounts of total suspended solids, phosphorus, nitrogen, biochemical oxygen demand (BOD), and metals from stormwater passing through the system. Due to these groundwater recharge and water quality benefits the EPA has listed PCP as a best management practice for stormwater control (EPA, 1999).

2.6 NOISE REDUCTION

Previous research has indicated that the structure of PCP results in reduced road noise from vehicle travel. Olek et al. (2003) investigated the sound characteristics of PCP under varying tire speeds and vehicle types, results were compared to traditional dense asphalt. The results indicated that PCP reduced noise from 3% to 10% compared to traditional asphalt pavements. The noise reduction was attributed to open pores at the surface allowing sound waves to enter the concrete matrix and dissipate energy (Olek, 2003). Additional research compared the noise characteristics of PCP to several other paving materials, including traditional concrete pavements and asphalt. The results indicated that pervious concrete would consistently produce the lowest noise out of any paving material tested (Kajio et al., 1998). As with the previous study the noise reduction was attributed to the structure of pervious concrete reducing noise generation at the interface between the road surface and the tire.

2.7 SKID RESISTANCE

Previous studies have indicated that PCP also allows for increased skid resistance when compared to traditional concrete (Tennis et al, 2004 and Ferguson, 2005). Rougher surface and the presence of pores to remove stormwater during the summer and melt water during the winter were found to create drier surfaces resulting in better grip between tires and the pavement surface.

2.8 TEMPERATURE BEHAVIOR

Both PCP and pervious asphalt pavements have temperature characteristics that differ from traditional pavements in two ways; (1) reduced heat island effect in warm conditions, and (2) warmer subsoil temperatures during the winter season. These characteristics influence how pervious pavements will perform in urban environments and in areas where seasonal temperatures can damage pavement materials through freezing and thawing cycles.

The heat island effect occurs when darker materials absorb heat from sunlight during the day reaching temperatures higher than the ambient air temperature; heat is then radiated during the night when the air temperature cools (Cambridge, 2005). Heat radiated from these surfaces can make living in urban areas unpleasant and can increase the temperature of stormwater entering nearby water bodies. PCP has two characteristics that have been cited as reducing the heat island effect. The void structure allows air to infiltrate through the material, allowing pervious concrete to radiate more heat during the day and lowering the temperature of the pavement (PCA, 2003). PCP has also been noted to have a higher albedo than traditional pavements resulting in less heat being absorbed when exposed to sunlight (PCA, 2003).

Seasonal variations in temperature have an effect on the performance of pavement materials. Freezing and thawing of water in the subsurface, also known as frost heave can cause significant damage to a pavement and result in a reduced service life. The extent of frost heave under a pavement surface is highly dependent on the temperature profile of the subsoil. Blackstrom (2000) investigated the temperature profile of subsurface material under a traditional pavement surface and a pervious pavement surface. The results of the investigation indicated that pervious pavement systems have a lower depth of frost penetration and reduced frost heave compared to traditional pavements. This is due to higher groundwater levels under pervious pavements; the increased amount of water results in a large thermal reservoir that is able to release heat over time and delay the onset of freezing. However, the research did find that pervious pavements freeze sooner than traditional pavements due to pore spaces increasing the contact with ambient air. Blackstrom's investigation involved pervious asphalt pavements; however, similar effects to the groundwater table and freezing profile are expected for PCP.

2.9 STRENGTH

Due to the open structure of PCP they typically have lower strength and durability when compared to traditional concrete pavements (Schaefer, et al., 2006; Chopra et al., 2007). However, several studies have shown that adequate strength can be achieved for a variety of applications in which pervious pavements would be useful, specifically low-volume traffic areas such as parking lots, driveways and sidewalks (Ghafoori and Dutta, 1995a). In these areas the strength values for PCP would be sufficient to meet structural demands while providing benefits to stormwater control and treatment.

PCP relies on the interaction of the cement paste and aggregates for strength (Chindaprasirt, 2008). Due to the structure of PCP, force is transferred through the cement paste to the aggregate when loads are applied. The cement paste layer surrounding the aggregate is typically thin; to fully develop strength sufficient paste must be present so that failure occurs through the aggregate. Increasing the bond between the cement paste and the aggregate, increasing the amount of aggregate present or a combination of both is needed to strengthen pervious concrete.

Several laboratory investigations have been conducted to determine the 28-day compressive strength of PCP. These values are used as a general indication of the strength of pervious concrete and there is a strong correlation between compressive strength and flexural strength (Ghafoori and Dutta, 1995a). Typical compressive strength values for pervious concrete are reported to be about 2,500 psi with the values ranging from a lower bound of 500 psi to an upper bound of 4,000 psi in the literature (Tennis, 2004). This wide variation in compressive strength is attributed to several factors including compaction energy/densification, water to cement ratio (w/c), aggregate to cement ratio (a/c), aggregate size/type, and presence of admixtures (Chindaprasirt, 2008; Ghafoori and Dutta, 1995a).

The amount of compaction energy impacted onto pervious concrete has a direct correlation to the unit weight of a pervious concrete specimen (Ghafoori and Dutta, 1995a). Several compaction techniques have been used by various researchers, all have indicated increasing amounts of compaction energy increases the unit weight of pervious concrete and therefore compressive strength. The relation between compaction energy and compressive strength is non-linear; with greater incremental increases in compressive strength at low compaction energies and almost no incremental increase at compaction energies above 5,000 ft-lbs/ft^3 (Ghafoori and Dutta, 1995a).

The ratio of w/c ratio has a large and complex impact on the overall strength of pervious concrete (Meininger, 1988). Several studies have identified the optimal w/c ratio to range from 0.30-0.45, with values lower or higher than this resulting in lower compressive strength values (Meininger, 1988; McCain and Dewoolkar, 2010). Low w/c ratios result in reduced workability and hydration of the cement paste, resulting in poor bonding between the cement paste and the aggregate. When this bond is poor the pervious concrete will fail through the thin cement paste and not through the aggregate as intended. Higher w/c ratios result in better workability; however, the cement paste can become fluid and not adhere to the aggregate. The resulting cement paste pools near the bottom of the PCP reducing strength and creating an impermeable layer preventing infiltration.

Aggregate to cement (a/c) ratio also has an impact on the strength properties of PCP (Ghafoori and Dutta, 1995). Mix designs with a lower a/c ratio (4:1) were found to be substantially stronger than mix designs with higher a/c ratios (6:1) with all other variables held constant. This difference is likely due to the increased amount of cement paste available to coat and connect aggregate particles, providing a more rigid bond between aggregate particles and the cement paste (Chopra et al., 2007).

Compressive strength testing of pervious concrete is derived from methods used in traditional concrete testing. ASTM C192 (2010), *Standard Practice for Making and Curing Concrete Test Specimens* has been used to prepare cylinders (McCain and Dewoolkar, 2010; Schafer et al., 2006; Meininger, 1988). Compressive strength has been primarily been determined using ASTM C39 (2010), *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimen* using specimens with a 2:1 height to diameter ratio. Although there are no standards for compressive strength testing of pervious concrete, various specimen characteristics including height to diameter ratio, specimen size, and end capping methods have been found to affect the compressive strength measurements of traditional concrete (Klieger and Lamond, 1994). The effect of specimen size on compressive strength testing was investigated by McCain and Dewoolkar (2010); specimens with a height to diameter ratio of 2:1 were cast with diameters of 3", 4" and 6". Specimens with diameters of 4" and 6" resulted in similar compressive strength values, whereas specimens 3" in diameter exhibited higher compressive strength values. To conserve material and simplify transportation of specimens, the authors recommended specimens be 4" in diameter with a 2:1 height to diameter ratio when using pervious concrete with aggregate 3/8" in size. End preparation methods such as grinding and sulfur capping were investigated by Rizvi et al. (2009). Specimens from several mix designs were prepared using both procedures and tested for compressive strength. End grinding was found to be slightly more consistent than sulfur capping for the pervious concrete mixes evaluated. The authors noted that several more mix designs should be evaluated and rubber capping should also be included in future analysis.

2.10 HYDRAULIC CHARACTERISTICS

Numerous researchers have investigated the hydraulic properties of PCP in the field and in the laboratory. A majority of the laboratory studies used to determine the hydraulic conductivity of pervious concrete employed a falling head or a constant head permeameter adapted from soils testing. Typical values for the hydraulic conductivity of PCP are reported to be between 280 in/hr to 1,730 in/hr (NRMCA, 2004).

Meininger (1988) conducted an investigation into the hydraulic characteristics of PCP using a falling head permeameter; the goal of the research was to determine how permeability was affected by void spaces. Results indicated that in pervious concrete with less than 15% void spaces, voids are not connected, resulting in little or no values for hydraulic conductivity; above this value, permeability increases exponentially with void content. Several subsequent researchers have observed this relation between void content and hydraulic conductivity.

Ghafoori and Dutta (1995a) studied the effects of a/c ratio and compaction energy on the hydraulic conductivity of PCP specimens in the laboratory. Both factors affected hydraulic conductivity values of PCP. Increasing compaction energy corresponded to lower hydraulic conductivity values; this was attributed to higher compaction energy compacting the concrete resulting in fewer void spaces for water to infiltrate. Increasing a/c ratio resulted in higher hydraulic conductivity values when all other factors were kept constant. Higher a/c ratios resulted in larger and more frequent void spaces, allowing larger amounts of water to pass through the specimen.

Montes and Haselbach (2006) investigated saturated flow in PCP to determine if laminar flow conditions could be assumed and Darcy's law be applied to PCP studies. Using a falling head permeameter and measuring outflow they were able to show that laminar flow exists in pervious concrete under most testing conditions and the use of Darcy's law is acceptable. The researchers applied the Carman-Kozeny equation to model the relation between porosity and saturated hydraulic conductivity. Results showed that the model based on the Carman-Kozeny equation was able to predict the saturated hydraulic conductivity using information on the porosity with relative accuracy.

Sansalone et al. (2008) investigated the pore structure of pervious concrete specimens using x-ray tomography. Based on the analysis the researchers identified several other characteristics that affect the hydraulic properties of pervious concrete. Total porosity was measured and compared to effective porosity (all pores that connect the top and bottom surfaces and can transport water) the researchers found that at values below 15% porosity most pores are separate and do not connect to one another or to the ends of the specimen resulting in no permeability.

McCain and Dewoolkar (2010) investigated the effects of specimen size on the hydraulic conductivity of PCP using a falling head permeameter. Specimens with diameters of 3", 4" and 6" and a constant depth of 6" were tested for hydraulic conductivity. Results indicated that 4" and 6" specimens provided similar results while 3" specimens showed reduced hydraulic conductivity. Based on these results the authors recommended using 4" x 6" cylinders for hydraulic conductivity testing. The research also investigated the effect of w/c ratio on the hydraulic conductivity of pervious concrete by examining mix designs with w/c ratios of 0.25, 0.29 and 0.33. Hydraulic conductivity values were found to decrease with increasing w/c ratios; the effect on hydraulic conductivity was explained by specimens with low w/c ratios having a lower amount of cement paste, resulting in less efficient densification and larger void spaces.

The field infiltration rate of PCP has been investigated using single ring, double ring, and falling head infiltrometers modified from soils testing (Bean et al., 2007; Henderson et al., 2009). Results from various PCP facilities showed infiltration rates ranging from 5 in/hr to 2,750 in/hr with areas of visible clogging reporting much lower infiltration rates when compared to areas that showed no signs of clogging. ASTM C1701 (2010), *Standard Test Method for Infiltration Rate of in Place Pervious Concrete*, has been developed to standardize the testing of infiltration rate of PCP in the field. Field observations have indicated that clogging of PCP by sand and organic matter can severely reduce infiltration rates (Henderson et al., 2009). This effect has been modeled in the laboratory using a uniformly graded sand to clog pervious concrete specimens. In these studies hydraulic conductivity was found to reduce by 30%-40% due to clogging (Joung and Grasley, 2008; Deo et al. 2010). Regular cleaning of PCP facilities is recommended to prevent the buildup of material in the pores of PCP (ACI, 2010)

2.11 VOID RATIO

Void ratio is a measure of the total open space within the pervious concrete. It is a comparison of the volume of voids, to the total volume of cement paste and aggregate. Typical void ratio for pervious concrete is 18-35% (ACI, 2010; Tennis et al., 2004). This range is considered ideal to provide enough strength, while allowing for sufficient hydraulic conductivity. Void ratio was shown to increase with a decrease in aggregate to cement ratio (Park & Tia, 2004). As the amount of cement covering each aggregate increases, the voids in pervious concrete are filled, reducing void ratio. It has been shown that void ratio increases as the unit weight decreases (Wang et al., 2006). With an overall denser sample, and a consistent density of aggregates and cement, the result is a lower void ratio.

2.12 DURABILITY IN COLD WEATHER CLIMATES

2.12.1 Freeze-Thaw

Due to the open pore structure and thin cement paste there is concern about the ability of pervious concrete to resist cold weather climates due to freeze-thaw cycles and the application of deicing salts. Traditional concrete pavements resist freeze-thaw cycles by entraining air within the concrete. Air entraining admixtures added during construction create 4% to 8% air content in the concrete in the form of independent microscopic pores. These pores provide space for water to expand during freezing cycles; this reduces the overall hydraulic forces on the concrete preventing fracture. Pervious concrete has a much larger void system; typically 15-30% is needed to achieve the required permeability. Under normal conditions water passes through these voids into an underlying layer to be infiltrated or collected for discharge. If this pore space is saturated when freezing occurs, then the expanding water will stress the cement paste that bonds aggregate, leading to aggregate becoming dislodged. Although saturation such as this is not common in the field the National Ready Mixed Concrete Association (2004) cites conditions that can lead to complete saturation of PCP. Complete saturation can occur in the field when pores become clogged with sand or debris preventing water from draining, or when high groundwater tables result in moisture flow into the PCP (NRMCA, 2004). Saturated freezing can be prevented by several methods; (1) properly constructing the pervious concrete lot to have a large gravel subbase that extends beyond the frost line of the soil, (2) adding under drain piping in the gravel subbase to transport away excess water, and (3) regular cleaning of the pervious concrete to prevent the accumulation of clogging fines.

2.12.2 Test Method

The large void spaces and thin cement paste leave pervious concrete susceptible to freeze-thaw type damage, an issue that has limited its use in cold regions. The presence of water, by design, puts pervious concrete in a vulnerable state. When fully saturated in water and frozen, the water expands forcing the aggregates apart. The standard test for freeze-thaw durability, ASTM C666 (2010) *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*, consists of cycling fully saturated concrete specimens 7 times a day, until 300 cycles. Mass lost of the samples is then measured, with 15% loss considered as failure (Schaefer et al., 2006). Tests have shown that with the addition of sand, pervious concrete can withstand over 300 freeze-thaw cycles, passing the ASTM C666 standardized test for durability (Kevern et al., 2008b). Other investigations have studied adding admixtures and fibers or changing the water to cement ratio (w/c), coarse aggregate, and moisture conditions (Ghafoori & Dutta, 1995; Kevern et al., 2008a; Schaefer et al., 2006; Wang et al., 2006; Yang et al., 2006; Yang, 2011).

The American Concrete Institute committee 522 report does not recommend the ASTM C666 test, because the test does not represent field conditions well (ACI, 2010). The fully saturated test condition and the rapid cycling of freeze-thaw make for an unrepresentative testing environment. As an alternative, testing under drained condition and one freeze-thaw cycle per day has been recommended by some researchers (Olek & Weiss, 2003; Yang, 2011). It has been suggested that increased saturation conditions are needed for damage and that below the critical saturation levels, no damage would occur in pervious concrete from freezing and thawing (Yang et al., 2006). Critical saturation in conventional concrete is expected to be about 60% for the freeze-thaw damage to occur (Litvan, 1973; Vuorinnen, 1970). Additionally, frozen water in the large pores of pervious concrete acts to create negative vapor pressures, drawing the liquid water through the cement paste, causing scaling damage (Harnik et al., 1980).

2.12.3 Damage

Damage done during freeze-thaw cycles in concrete is typically one of the following: internal paste deterioration, surface scaling, and D-cracking (ACI, 1992). Surface scaling, the loss of paste or mortar from the surface of the concrete, is the most common damage, and typically removes layers less than 1 mm (ACI, 1992). D-cracking, which is characterized as internal failure in a nondurable aggregate generally occurs near the edges and joints, and is a result of expansion in the aggregate (Sawan, 1987). Internal paste deterioration typically occurs from the internal pressure in the pore structure that generates when freezing occurs during critical saturation (Pigeon, 1994).

2.12.4 Durability

The type of materials, their properties, and the ratios at which they are included in the construction of pervious concrete can have significant effects on the freeze-thaw durability as well. Kevern, et al. (2010) suggest that the key factor for freeze-thaw durability is the aggregate absorption, and recommend absorption of less than 2.5% by sample mass for high durability mixes. The addition of sand has been shown to improve freeze-thaw durability in rapid freeze-thaw testing (Schaefer et al., 2006). Increased water to cement ratio has been shown to improve freeze-thaw durability in slow freeze-thaw testing in water cured samples (Yang, 2011). The use of air entraining admixtures has been shown to both improve freeze-thaw durability in rapid cycles (Kevern et al., 2008a; Yang et al., 2006) and have no effect in slow cycles (Yang, 2011). Kevern, et al. (2008a) reported that by adding fibers freeze-thaw durability and workability can be improved without sacrificing infiltration potential.

2.12.5 Effects of Deicing Salts

In cold climates, road salts are used to melt snow and ice on pavements. The commonly used salts are sodium chloride and calcium chloride. Salt exposure in concrete can lead salt crystals to form in the pores, and at high concentrations can change the chemical composition in the cement paste (Darwin et al., 2007). The chemical reaction causes the cement paste to lose its structure, and the bonds can be destroyed (Cody, et al., 1996; Lee, et al., 2000).

Studies have shown that a 2-4% solution of salt causes maximum scaling (cement paste to be dislodged) in saturated conditions, and that above and below this range less scaling is expected (Marchand et al., 1999; Verbek and Kleiger, 1957). Conversely, for the wetting-drying condition, the amount of damage increases as the concentration of salt increases (Cody et al., 1996). Freeze-thaw testing conducted with a 3% sodium chloride solution also showed that as the solution freezes, the concentration of the unfrozen solution can rise to nearly 4 times the original concentration (Chan et al., 2007). The effect, known as freeze concentration, is believed to aid in the process of supercooling. Supercooling occurs when the freezing point of the solution is depressed because of the salt concentrations, until the point where the phase shift in the water does occur, and at much larger pore pressures (Harnik, et al., 1980).

Harnik, et al. (1980) state that the application of deicing salts allows the degree of saturation in conventional concrete to exceed the amount normally attainable with pure water.

Additionally salt crystallization is identified as a source of pressure in the large pores in concrete, by both physical forces and hydraulic pressures, as it draws water out of the smaller pores.

Pigeon and Pleau (1995) have shown that in the ASTM C666 rapid freeze-thaw testing the use of air entraining admixtures can significantly improve the deicing scaling resistance. Yang (2011) however showed that in a slower, one cycle per day testing, no increase in durability was seen; and suggests it may be due to the additional air voids becoming saturated in the longer freeze-thaw cycles.

CHAPTER 3

EVALUATION OF STRENGTH AND HYDRAULIC TESTING METHODS OF PERVIOUS CONCRETE

3.1 ABSTRACT

Common methods for compressive strength and hydraulic testing were investigated for pervious concrete with 3/8" (0.95 cm) aggregate. Rubber capping is simpler and yielded more consistent compressive strength measurements than sulfur capping, and is recommended. Height to diameter ratio (H:D) of a minimum of 1.5:1 is recommended for compression strength testing. Measured strengths on specimens with H:D < 2 may be reduced by 10% to estimate strength for H:D of 2:1. The hydraulic conductivity correlated to the infiltration rates from single ring, double ring and falling head infiltrometers as 1.0 : 1.8 : 1.5 : 9.0 for 6” (15.2 cm) thick pervious concrete. The double ring infiltrometer provided results only for infiltration rates less than 150 in/hr (0.1 cm/s). If the falling head infiltrometer is used because of its easier testing procedure, its infiltration rate can be divided by 5 to estimate infiltration rate of the single ring infiltrometer.

3.2 INTRODUCTION

Pervious pavement systems have been proposed as a method to meet increased stormwater regulations while still providing for structural needs (ACI, 2010; Ferguson, 2005). Conventional pavements can collect pollutants and prevent stormwater infiltration into the subsurface resulting in large amounts of runoff requiring additional treatment facilities. Pervious pavements allow stormwater to pass through the surface material; this reduces runoff and facilitates removal of contaminants such as nutrients and heavy metals (Ferguson, 2005). In addition to improving water quality, pervious pavements have several other advantages over traditional pavement materials including (i) reduced noise from vehicular traffic, (ii) reduced heat island effects, (iii) improved skid resistance during storm events, and (iv) faster snowmelt characteristics (Ferguson, 2005). Despite these benefits, potential of lower compressive strength, clogging and susceptibility to freeze-thaw damage has limited the use of pervious pavements, especially in cold environments (Schaefer et al., 2006).

This paper focuses on pervious concrete pavement (PCP) and measurements of their compressive strength and infiltration capacity. PCP is made in a manner similar to traditional concrete pavements; with water, cement, aggregate and admixtures combined to create PCP. The primary difference between traditional concrete pavements and PCP is the complete elimination or drastic reduction of fine aggregates (sand) and the use of a uniformly graded coarse aggregate. These two characteristics are expected to create large and continuous voids. PCP has been used successfully in several areas including the Europe, Japan, and the southern United States; however, PCP is limited in cold weather regions that experience repeated freezing and thawing conditions (Tennis et al., 2004). Laboratory studies have been performed to create mix designs that meet the mechanical and hydraulic needs of PCP in the field (Schaefer et al., 2006; Meininger, 1988; Ghafoori and Dutta, 1995a). Compressive strength of pervious concrete has been found to range from 500 psi to 4,000 psi (3.4 MPa to 27.5 MPa); and is influenced by factors such as compaction energy/densification, water to cement (w/c) ratio, aggregate to cement (a/c) ratio, aggregate size/type, and presence of admixtures (Tennis et al., 2004; Ghafoori and Dutta; 1995a; Chindaprasirt et al., 2008).

Procedures for measuring the compressive strength of conventional concrete are well established. ASTM C39 (2010) provides procedures for determining compressive strength of concrete and guidelines on how to interpret the results. Previous studies have evaluated factors such as height to diameter ratio, sample size, diameter to aggregate ratio and capping conditions for their effects on the compressive strength measurement of conventional concrete (Klieger and Lamond, 1994; Head et al., 1999).

PCP is a relatively newer product as compared to conventional concrete and the effects of various testing parameters on compressive strength testing of PCP are less well understood. Rizvi et al. (2009) noted that the course surface of PCP requires the use of end preparation to prevent uneven loading on the surface of cylindrical strength samples. They examined how different compaction procedures and capping conditions affected compressive strength testing of PCP. Test cylinders were prepared with different compaction methods and capped with sulfur compound or end ground to determine which method provided more consistent compressive strength measurements. Their results indicated that the compressive strength measured on the end ground specimens yielded slightly lower standard deviation than sulfur capped specimens; however the authors recommended further testing.

McCain and Dewoolkar (2010) investigated the effects of sample size on compressive strength measurements of PCP. Cylindrical specimens with diameters of 3”, 4” and 6” (7.6 cm, 10.1 cm and 15.2 cm) were cast with a 2:1 H:D ratio. Specimens with diameters of 4” and 6” (10.1 cm and 15.2 cm) were found to provide similar results, while 3” (7.6 cm) specimens resulted in higher compressive strength measurements at the same unit weight. Based on this finding the authors recommended using 4” (10.1 cm) diameter specimens for testing compressive strength of PCP. They preferred 4” (10.1 cm) compared to 6” (15.2 cm) because the smaller size reduced the amount of material required and were generally easier to handle. These tests were conducted on PCP made using a 3/8” (0.95 cm) aggregate and the authors noted that larger specimens might be needed for PCP incorporating aggregate larger than 3/8” (0.95 cm). It appears that the effects of H:D ratio and the effects of aggregate to specimen diameter ratio on the measurement of compressive strength of PCP have not been investigated.

Since for practical reasons PCPs are generally restricted to 3/8” (0.95 cm) aggregate size, McCain and Dewoolkar (2010) recommend using a minimum of 4” (10.1 cm) diameter specimens. However, there is still a need to evaluate if a H:D ratio of less than 2:1 can be used. In general, field PCP thickness is targeted to be 6” (15.2 cm) but could vary from 4” to 8” (10.1 cm to 20.3 cm). Therefore for field cores of 4” (10.1 cm) diameter (per McCain and Dewoolkar (2010) recommendation), PCP specimens with H:D ratios ranging from 1:1 to 2:1 are of interest.

Ensuring that water can infiltrate through PCP is necessary to meet stormwater control requirements. Hydraulic conductivity of PCP has been determined in the laboratory using constant head or falling head methods adapted from soils testing (Schaefer et al., 2006; Meininger, 1988; Ghafoori and Dutta, 1995a). Typical values for hydraulic conductivity of PCP range between 280 in/hr to 1,680 in/hr (0.2 cm/sec to 1.2 cm/sec) (NRMCA, 2004). Hydraulic conductivity of PCP has been found to be highly dependent on void ratio, with at least 15% voids needed for sufficient hydraulic conductivity values, below this value voids are present but they may not be continuous and result in zero permeability (Meininger, 1988). Montes and Hasselbach (2006) examined the flow conditions present during hydraulic conductivity testing of PCP. Their results indicated that under head values and flow rates commonly used for laboratory testing laminar flow conditions exist and Darcy’s law is valid. Sansalone et al. (2008) examined the pore structure of PCP using x-ray tomography and determined that several factors including pore connectivity, tortuosity of pore space and size of pores influence the hydraulic properties of PCP. McCain and Dewoolkar (2010) investigated the effects of specimen size on hydraulic conductivity measurement by casting specimens with diameters of 3”, 4” and 6” (7.6 cm, 10.1 cm and 15.2 cm). The heights of all specimens were kept constant at 6” (15.2 cm) to represent the typical thickness of PCP in the field. Specimens 4” and 6” (10.1 cm and 15.2 cm) in diameter resulted in comparable values, while samples 3” (7.6 cm) in diameter resulted in smaller hydraulic conductivity measurements. As with compressive strength testing the authors recommended using specimens 4” (7.6 cm) in diameter for hydraulic conductivity testing.

Additional studies have been conducted to monitor infiltration rates of PCP in the field (Bean et al., 2007; Chopra and Wanielistra, 2007; Henderson et al., 2009). Typically, single ring infiltrometer and falling head infiltrometer methods adapted from soils testing were used to determine infiltration rates, Infiltration rates of PCP in the field have been found to range from 11 in/hr to 2,700 in/hr (0.007 cm/sec to 1.9 cm/sec) based on mix design and testing method used. Measurements of saturated hydraulic conductivity in the field have been conducted by Chopra et al. (2010); however, the method is destructive or requires placement of specialized equipment during construction. ASTM C1701 (2010), *Standard Test Method for Infiltration Rate* *of in Place Pervious Concrete*, has been developed recently as a method to determine infiltration rate of PCP in the field. Attempts at establishing correlations between field infiltration rates and hydraulic conductivity values determined in the laboratory for PCP were not found in the literature.

3.3 RESEARCH SIGNIFICANCE

The above literature review reveals that although testing for compressive strength and hydraulic parameters of PCP has been conducted, the testing methods remain largely not standardized. Most compressive strength testing methods are adapted from testing of conventional concrete and the effects of factors such as the H:D ratio of test specimens and capping condition are unknown for PCP. Infiltration rates of PCP have been determined by many methods and it appears that there is no literature examining comparisons among different methods and the relation between infiltration rate and hydraulic conductivity for PCP. Therefore, this investigation examined the influence of the above factors (H:D ratio and capping condition) on compressive strength testing and the relationship among different field infiltration measurements and hydraulic conductivity of PCP.

3.4 EXPERIMENTAL INVESTIGATION

The specific objectives of this experimental investigation were to determine for PCP: (1) the effects of sulfur capping and rubber capping on the compressive strength measurement; (2) the effects of height to diameter (H:D) ratio on the compressive strength measurement; (3) the relation among different methods (single ring infiltrometer, double ring infiltrometer, and falling head infiltrometer) used to measure infiltration rate in the field; and (4) how field methods used to measure infiltration rate compare to hydraulic conductivity measurements in the laboratory.

3.4.1 Field Testing

Two PCP facilities located in Vermont were selected for comparing commonly used methods for the measurement of infiltration rate. The first facility is located on College Street in Burlington, Vermont, near the Lake Champlain waterfront and was constructed in June 2009. The second facility is located at Heritage Flight near the Burlington International Airport in South Burlington, Vermont, and was constructed in September of 2009. Both facilities consist of a 6” (15.2 cm) layer of PCP with 34” (86.3 cm) gravel subbase. Five sites were selected for testing at the College Street facility and ten sites were selected at the Heritage facility. At each site infiltration rates were determined using single ring, double ring and falling head infiltrometer methods.

3.4.2 Mix Designs and Specimen Preparation

A summary of mix designs used in this study is provided in Table 3.1. Mixes included; three mixes reported by McCain and Dewoolkar (2010) based on the Randolph Park and Ride PCP in Randolph, Vermont (identified with prefix RAND in Table 3.1); four mix designs based on the field mix used at the College Street and Heritage facilities (prefix HERT); and six mix designs developed as part of a separate investigation into using recycled concrete aggregate (RCA) in PCP (prefix RCA). The mix design used at the College Street and Heritage facilities is listed in Table 3.1 as FIELD and was recreated for laboratory testing with HERT 2.

Mixes derived from the Randolph Park and Ride consisted of a 3/8” (0.95 cm) crushed ledge aggregate from West Lebanon, New Hampshire; Lafarge type I-II cement; a viscosity modifying admixture (V-MAR3); an air entraining admixture (Daravair); a high-range water reducer (ADVA 190); and a stabilizer (Daratard). In this study w/c ratio was varied across three RAND mix designs to develop different mixes. Mixes based on the field facilities consisted of a 3/8” (0.95 cm) crushed ledge aggregate from Winooski, Vermont; Lafarge type I-II cement; an air entraining admixture (Micro Air); a high-range water-reducing admixture (Glenium 7500); a viscosity modifying admixture (VMA 362) and different quantities of class F fly ash. The effects of fly ash in PCP will be discussed elsewhere. Mix designs incorporating RCA used the same aggregate, cement and admixtures with the exception of fly ash. RCA (3/8” size [0.95 cm]) replaced virgin aggregate in varying amounts based upon the mix design. The effects of RCA on PCP are presented in another paper, in this study these specimens were used to populate the data set used to study the effects of capping condition and H:D ratio on the compressive strength measurement (Berry et al., 2012). The specific proportions used in each mix design are summarized in Table 1. As seen, a broad range of mixes were investigated in this study.

HERT and RCA mixes were selected to evaluate the effects of capping condition on compressive strength measurement of PCP. To evaluate the effects of H:D ratio on compressive strength testing specimens were prepared with H:D of 2:1, 1.5:1 and 1:1 using all mix designs listed in Table 1 with the exception of 1:1 specimens for RCA mix designs. H:D ratios greater than 2:1 were not evaluated in this study because for field cores of 4” (10.1 cm) in diameter, H:D ratios would rarely be greater than 2. All samples had a diameter of 4” (10.1 cm) resulting in heights of 8” (20.3 cm) 6” (15.2 cm) and 4” (10.1 cm) for 2:1, 1.5:1 and 1:1 H:D ratios respectively. The procedure described by Schaefer et al. (2006) was used to prepare the mix designs listed above. Cylindrical specimens were cast in accordance with ASTM C192 (2010), *Practice for Making and Curing Concrete Test Specimens in the Laboratory*. All specimens were cast as cylinders 4” (10.1 cm) in diameter with a H:D ratio of 2:1. For 1.5:1 H:D specimens two inches of material was trimmed, alternating between the top and bottom of a 2:1 H:D specimen. Cutting a 2:1 H:D specimen in half created two 1:1 H:D specimens. This procedure ensured that all specimens received similar amounts of compaction energy regardless of their H:D ratio. Samples were washed thoroughly after cutting to remove any debris from cuttings trapped in pores.

Slab samples were prepared using all three RAND mixes along with HERT 1 and 2 mixes to test infiltration measurement methods in the laboratory. Slabs were 28” (71.1 cm) long by 24” (60.9 cm) wide by 6” (15.2 cm) thick. This thickness was selected to represent the thickness of typical PCP in the field. The width and length dimensions of the slabs were selected to prevent water that may flow horizontally and exit along the sides of the slab. Slab specimens were cast into wooden molds and compacted in a manner similar to field placement, and allowed to cure for 7 days under a plastic covering. After 7 days slabs were removed and tested for infiltration rate. Cores were removed from the slabs after infiltration testing had been completed. Three cores 4” in diameter were removed from the central portion of each slab for subsequent hydraulic conductivity testing.

3.4.3 Compressive Strength

Compressive strength testing was conducted in accordance with ASTM C39 (2010), *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. ASTM C1231 (2010), *Standard Practice for Use of Unbonded Caps in Determination of Compressive*

Strength of Hardened Concrete Cylinders, was followed for rubber capped specimens and sulfur capped specimens were capped in accordance with ASTM C617 (2010), *Standard Practice for Capping Cylindrical Concrete Specimens*. Eight specimens were tested for compressive strength for each mix design and H:D ratio investigated.

3.4.4 Infiltration Testing Methods

Three methods to determine the infiltration rate of PCP were investigated; a single ring infiltration method described in ASTM C1701 (2010), *Standard Test Method for Infiltration Rate of In Place Pervious Concrete*; a double ring infiltration method, modified from the ASTM C1701 procedure; and a falling head infiltrometer developed by the authors. The University of Vermont and the Vermont Agency of Transportation began using the falling head infiltrometer method for long term field monitoring before the publication of ASTM C1701; therefore this method was included to determine how it correlated to the ASTM testing method.

Infiltration rate per single ring infiltrometer was determined in accordance with ASTM C1701. The apparatus, shown in Figure 3.1a, consists of a ring 12” (30.0 cm) in diameter, which is to be sealed to the PCP surface with plumber’s putty. The location was pre-wetted by pouring 0.12 ft^3 (3.6 liters) of water into the ring and keeping water levels between 0.4” and 0.6” (1.0 cm and 1.5 cm) above the surface of the pavement. Locations taking more than 30 seconds to infiltrate the pre-wetting water were tested for infiltration rate using 0.12 ft^3 (3.6 liters) of water, other locations required 0.63 ft^3 (18 liters) of water for testing. Water was poured into the ring and kept between 0.4” and 0.6” (1.0 cm and 1.5 cm) above the surface of the pavement. Time was recorded from when water made contact with the pavement surface to the time when water was no longer visible. Tests were repeated five times at each location to ensure consistent results. The equation provided in ASTM C1701 was used to calculate infiltration rate (Eq. 1 below). Because the testing equipment does not pass through the pavement material during testing saturated flow conditions are not present and constant head permeability equations are not valid; therefore, Eq. 1 determines infiltration rate by dividing the volume of water by the area tested and the time required for infiltration.

$$I = \frac{KM}{(D^2 * t)} \tag{1}$$

where

I = Infiltration rate, (in/hr),
M = Mass of infiltrated water, (lbs),
D = Inside diameter of infiltration ring, (in),
t = Time required for measured amount of water to infiltrate the concrete, (s),
K = Conversion factor 126,870 (in^3 s)/(lbs*hr)

The procedure adopted for determining infiltration rate with a double ring infiltrometer was similar to the single ring ASTM C1701 method with minor modifications. Pre-wetting was done in the same fashion; however, for infiltration testing a second ring 24” (60.9 cm) in diameter was placed around the inner ring of 12” (30.4 cm) in diameter as seen in Figure 3.1b. Water level was kept constant between the inner and outer rings at 0.4” to 0.6” (1.0 cm to 1.5 cm) above the pavement surface. If water levels could not be maintained between the inner and outer rings double ring infiltration testing was abandoned. The equation used with the single ring infiltrometer method (Eq. 1) was used with the double ring infiltrometer method.

The falling head infiltrometer device was designed and built at the University of Vermont as a simple and quick way to monitor the long term infiltration rate of PCP in the field. This method of measuring infiltration rate is similar to the method used by Henderson et al. (2009). A 2' x 2' x ¾" (60.9 cm x 60.9 cm x 1.90 cm) sheet of PVC acted as a base for the device, with a circular hole cut as a location to attach a standpipe. The standpipe was made using a 0.25" (0.63 cm) thick PVC pipe with 4" (10.1 cm) internal diameter. Milled viewports on the standpipe allowed for monitoring of water levels during testing. A ring of foam rubber was attached to the bottom of the device around the opening where the standpipe was located to create a seal between the infiltrometer and the PCP surface. Weights (about 120 lbs [55 kg]) were placed onto the device to compress the foam during testing as seen in Figure 3.1c.

The infiltration rate was measured by filling the standpipe and measuring the time for the water level to drop from 15" (38.1 cm) to 3" (7.6 cm) above the PCP. Because the area of pavement tested by the falling head infiltrometer was smaller than the other two methods, the falling head infiltrometer measurements were made at three locations and each location was tested three times. The resulting nine infiltration measurements were averaged and reported here. Infiltration rate was determined using Eq. 2 below.

$$I = \frac{c\left(h_1 - h_2\right)}{t} \qquad (2)$$

where,
I = Infiltration rate, (in/hr),
t = Recorded time, (s),
h_1 = Initial water level, (in),
h_2 = Finial water level, (in), and
c = Conversion factor = 3,600 s/hr.

3.4.5 Hydraulic Conductivity

After slab samples had been tested for infiltration rate using the above mentioned three methods, hydraulic conductivity measurements were conducted on cores taken from the slabs. Three cores were taken from the locations where infiltration measurements were conducted. Hydraulic conductivity was determined using a falling head permeameter developed by McCain and Dewoolkar (2010). The apparatus is shown in Figure 3.1d. Specimens were enclosed in a mold that was coated with a flexible rubber layer. This mold was secured with hose clamps to prevent water from flowing along the edge of the sample. Water was added to the downstream end of the device to expel any air voids that may have been present in the sample. Once all voids had been filled the water level was increased to 15" (38.1 cm) above the zero head value and allowed to fall to a height of 3" (7.6 cm), the time for this to occur was recorded. Montes and Haselbach[13] showed that under these head values laminar flow conditions are expected to exist in PCP and Darcy's law can be used to determine the hydraulic conductivity. Eq. 3 was used to determine hydraulic conductivity.

$$k = \frac{a\,L}{At}\ln\left(\frac{h_1}{h_2}\right) \qquad (3)$$

where,
k = Hydraulic Conductivity, (in/hr),

a = Cross-sectional area of the standpipe, (in^2),
L = Length of specimen, (in),
A = Cross-sectional area of specimen, (in^2),
t = Time for water to drop from h_1 to h_2, (hr),
h_1 = Initial water level, (in), and
h_2 = Finial water level, (in).

3.5 EXPERIMENTAL RESULTS AND DISCUSSION

3.5.1 Effects of Capping Condition

The results of testing performed to evaluate the effects of capping condition on the compressive strength measurement are summarized in Table 3.2. For the RCA mix designs (H:D ratio 1.5:1) four specimens were prepared using each capping method; and for the HERT mixes (H:D ratio 2:1) eight specimens were prepared for each capping condition. In general both capping methods resulted in similar average compressive strength values for a given mix design. However, standard deviation values for sulfur capped specimens were consistently higher than rubber capped specimens with the exception of HERT-1. For all mixes with the exception of HERT-4 and RAND-1 failure occurred through the aggregate. Most of the sulfur capped specimens failed due to shear fracture while rubber capped specimens failed in cone and shear fracture, or incomplete shear fracture through the top of the specimen. Differences in the fracture pattern are likely a result of the different capping materials used; the sulfur compound was able to enter the void structure during specimen preparation and bind the pervious concrete at both ends, this likely prevented the fracture surface from passing through the end of the specimen. The rubber pads were not able to bind the pervious concrete, allowing for fracture surfaces to occur through the end of the specimen.

Figure 3.2 shows the relation between compressive strength and unit weight values for the RCA and HERT mixes studied. As expected compressive strength was found to increase with unit weight, this trend has been noted in previous studies (Schaefer et al., 2006; McCain and Dewoolkar, 2010). Regression lines are plotted with R^2 values to indicate how well compressive strength correlated with specimen unit weight for a given capping condition. The 1.5:1 H:D RCA specimens showed little to no correlation to unit weight when capped with sulfur with an R^2 value lower than 0.2. Specimens capped with neoprene rubber pads showed a stronger relationship between unit weight and compressive strength (R^2 = 0.59) indicating that more consistent measurements can be obtained with this method. For 2:1 H:D HERT specimens both rubber capping and sulfur capping were found to result in consistent relationships between compressive strength and unit weight with R^2 values of 0.79 and 0.61 respectively. Sulfur capping was found to result in higher compressive strength values at a given unit weight compared to rubber capping with 2:1 H:D specimens. It is to be noted that for a particular mix design the specimens for sulfur capping as well as rubber capping were prepared using the same batch of the mix to eliminate any possible variations owing to specimen preparation.

Figure 3.3 shows the correlation between measurements of sulfur capped and rubber capped specimens. Compressive strength of samples capped with rubber are presented along the x-axis while results of sulfur capped samples are plotted on the y-axis for a given mix design. A 1:1 line is shown for comparison. Maximum and minimum values associated with average compressive strength values for rubber capped and sulfur capped specimens are shown as lines at each data point, with standard deviation values represented by intersections on the line. Regression lines are plotted to determine the overall relation between different capping methods.

Although there are some differences between 1.5:1 H:D and 2:1 H:D specimens, in general the averaged strengths were within 10% of the 1:1 line indicating that both methods provided similar measured compressive strength values.

Rubber capping produced more consistent compressive strength measurements for specimens with a H:D ratio of 1.5:1. For 2:1 H:D specimens both sulfur capping and rubber capping methods resulted in good correlations between sample unit weight and compressive strength over the range of compressive strength values observed. However sulfur capping resulted in increased compressive strength measurements and greater variability. In addition, due to the lengthy preparation work and potentially harmful chemicals involved with sulfur capping of pervious concrete, the use of unbonded rubber pads is recommended.

3.5.2 Height to Diameter Ratio

The effects of H:D ratio on compressive strength measurements were evaluated using RCA, HERT and RAND mixes; results are summarized in Table 3.3. Average unit weight values were consistent for H:D ratios of 2:1 and 1.5:1 for a given mix design; however 1:1 specimens exhibited slightly lower unit weight values in some mix designs. Standard deviations for compressive strength measurements were found to be similar for specimens with H:D ratios of 2:1 and 1.5:1 , but were consistently higher for 1:1 H:D ratio specimens.

Figures 3.4a-c show compressive strength plotted against unit weight for specimens from the various mix designs studied at different H:D ratios. For all mixes compressive strength generally increased with increasing unit weight. RCA and HERT mixes used similar materials and were found to have comparable compressive strength and unit weight values; as a result they fall in the same general region. RAND mixes exhibited a wider range of unit weight and compressive strength values, due to the large difference in w/c ratio among the various RAND mix designs. H:D ratio was found to consistently affect compressive strength measurements. For RCA and HERT mixes, 1.5:1 H:D specimens resulted in higher compressive strength values when compared to 2:1 H:D ratio specimens of the same unit weight. Both 1.5:1 and 2:1 H:D ratio specimens showed good correlation between specimen unit weight and compressive strength. RAND mixes showed a less clear relation between H:D ratio and compressive strength values. Compressive strength results for the 1:1 H:D specimens were consistently higher than the 1.5:1 H:D ratio specimens at a given unit weight; on the other hand, results of 2:1 H:D ratio specimens varied widely across the range of values. At a low w/c ratio the 2:1 H:D ratio specimens showed the lowest compressive strength measurement, as w/c ratio increased all H:D ratios showed similar compressive strength values. For both HERT and RAND mixes specimens with a H:D ratio of 1:1 were found to have poor correlations to unit weight as compared to specimens with H:D ratios of 1.5:1 and 2:1.

Figure 3.5a shows the relation between the average compressive strength measurements of 2:1 and 1.5:1 H:D ratio specimens. Maximum and minimum extents associated with each average value and standard deviation values are shown in a manner similar to Figure 3.3. Overall 1.5:1 H:D ratio specimens resulted in compressive strength measurements higher than 2:1 H:D ratio specimens. HERT mixes showed a 13% increase; RCA mixes showed a 14% increase and RAND mixes showed a 1% increase. The results of the RAND mixes might be influenced by the results of RAND-3, which deviates from the trend observed in all other mix designs. When all data are considered there is an average 10% increase to measured compressive strength values for 1.5:1 H:D ratio specimens compared to 8” (20.3 cm) specimens.

The relation between the average compressive strength measurement of 1:1 and 2:1 H:D ratio specimens is presented in Figure 3.5b. The correlation between 1:1 and 2:1 H:D ratio specimens was found to be less consistent compared to the relation between 1.5:1 and 2:1 H:D ratio specimens. For the HERT mixes, 1:1 H:D ratio specimens yielded 30% higher compressive strength when compared to 2:1 H:D ratio specimens. RAND mixes indicated that 1:1 H:D ratio specimens resulted in compressive strength measurements almost identical to the strengths obtained with 2:1 H:D ratio specimens. Averaging the results of HERT and RAND mixes results in 10% increase in the compressive strength measurement when 1:1 H:D ratio specimens are used. Therefore a correction factor of 1.1 is recommended to convert compressive strength of specimens with H:D ratios less than 2:1 to equivalent strength of 2:1 H:D ratio specimens. However, based on the difference in results between HERT and RAND mixes the relation between 1:1 and 2:1 H:D ratio specimens is considered to be less reliable; for this reason using the compressive strength measurement of 1:1 H:D ratio specimens to estimate the compressive strength of 2:1 H:D ratio specimens should be avoided if possible.

3.5.3 Field and Laboratory Infiltration

Figure 3.6 shows the results of field infiltration testing conducted at the College Street and Heritage facilities. Infiltration rates determined by single ring infiltrometer are presented along the x-axis, with double ring and falling head methods plotted along the y-axis. Due to the range of infiltration values observed the scale of the y-axis is larger than the x-axis in Figure 3.6. Regression lines are included to show the relation between the various methods investigated. All methods should result in zero infiltration when impermeable conditions exist, therefore the regression lines were forced through the origin. Not forcing the regression lines through the origin resulted in slope and R^2 values very similar to the ones from forced regression.

The falling head infiltrometer developed at UVM and the double ring infiltrometer showed linear relations to the measurements of the single ring infiltrometer. Results of the double ring infiltration method were 85% of those obtained using the single ring method, with a R^2 value of 0.99. This relatively small reduction to infiltration rate with the double ring method is likely a result of lateral flow being constrained by the infiltration of water in the outer ring. The falling head infiltration values were 4.7 times greater than the measurements of the single ring infiltrometer, with a R^2 value of 0.96. Single ring and field infiltrometer methods were found to be useful over a wide range of infiltration values; however, the double ring method was limited to areas where infiltration values were low enough (less than about 150 in/hr [0.11 cm/sec]) allowing a constant level of water to be maintained between the inner and outer rings.

The results of laboratory infiltration testing are compared to hydraulic conductivity in Figure 3.7. Hydraulic conductivity measured from laboratory cores is plotted along the x-axis while infiltration values found by single ring and field infiltrometer methods on the slabs where the cores for hydraulic conductivity testing were extracted are presented along y-axis. Regression lines and R^2 values show the relation between infiltration and hydraulic conductivity measurements. Double ring infiltration was not measured due to the high infiltration values of the slabs preventing the water level between the inner and outer ring to be kept at constant head value. Although lateral flow was not constrained in slab testing, no water was found to discharge from the sides of the slab.

Field methods used to determine infiltration rate correlated well with measured hydraulic conductivity values. The falling head infiltrometer reported infiltration rates 9.0 times higher (R^2 = 0.94) than the actual hydraulic conductivity of the slab, whereas the single ring infiltrometer resulted in values 1.8 times greater (R^2 = 0.95) than the hydraulic conductivity. Both methods indicated infiltration values greater than the saturated hydraulic conductivity measured in the laboratory, and results are likely a result of differences in the condition of the pervious concrete during testing. For infiltration testing pore spaces are unsaturated and water is able to fill these spaces with little resistance; however, in hydraulic conductivity testing the pervious concrete sample is saturated and water is present in the pore spaces at the beginning of testing. This difference in initial conditions of the pervious concrete has a considerable effect on the measured hydraulic characteristics.

The observed significant differences between the infiltration rates measured by the single ring infiltrometer versus the falling head infiltrometer are considered to be a result of the different head values used and the difference in geometry between the methods. The falling head infiltrometer method used higher head levels in the testing process compared to the single ring method, this higher head value probably increased the amount of lateral flow resulting in inflated infiltration values. Head values used in the single ring test method are much lower, and as a result less lateral flow occurs. Differences in the geometry of the testing methods could also account for the different infiltration values observed. The diameter of the standpipe used in the falling head infiltrometer (4" [10.1 cm]) is smaller than the diameter of the single ring infiltrometer (12" [30.5 cm]), as a result the ratio of perimeter to area being tested for infiltration is higher for the falling head infiltrometer compared to the single ring infiltrometer. If lateral flow is influenced by the ratio of perimeter available to area being tested, the falling head infiltrometer would have higher infiltration values, as was observed in this study.

Following the single ring infiltration testing, a circular area of about 16" (40.6 cm) in diameter was observed to be wet on the bottom of the PCP slabs. Recall that the diameter of the water column on the top of the slab was restricted to 12" (30 cm) in diameter, which is the diameter of the single ring infiltrometer. The head of water was about 0.4" (1.0 cm) to 0.6" (1.5 cm). In comparison, the flow of water was observed to spread from 4" (10 cm) in diameter from the top to about 22" (55 cm) in diameter at the bottom of the PCP slabs in the falling head infiltrometer testing with a variable head of water from 15" (`37.5 cm) to 3" (7.5 cm). These observations indicate that the lateral flow was indeed considerably greater in the falling head infiltrometer than in the single ring infiltrometer.

Although the lower head values of the single ring method resulted in reduced lateral flow, the required water levels (0.4" [1.0 cm] to 0.6" [1.5 cm]) were difficult to maintain compared to the falling head infiltrometer (15" [38.1 cm] to 3" [7.62 cm]). Both methods showed consistent trends between infiltration rate and actual hydraulic conductivity, with R^2 values of 0.95. Figure 3.8 compares infiltration measurements from the single ring method to the falling head infiltrometer method for both laboratory and field locations. The correlation between the two methods appears to be consistent over a wide range of infiltration values, indicating that the falling head infiltrometer can be used to predict infiltration values determined from the single ring method. It is to be noted that the measurements presented in Figure 3.8 showed all data points resulting in an average correlation of 4.94.

The falling head infiltrometer used in this investigation required less preparation and was simpler to operate than the single ring method. Both methods can be used to estimate one dimensional hydraulic conductivity value of PCP based on field measurements. The trends observed in this study were for high hydraulic conductivity values; care should be taken when applying this relation to low hydraulic conductivity situation such as clogged PCP. Additionally all field and laboratory hydraulic tests in this study were performed on PCP 6" (15.2 cm) thick, and for aggregate size of 3/8” (0.95 cm) different pavement thicknesses and aggregate sizes could result in different relations to those established in this study. Also the correlations involving the falling head infiltrometer are valid only for the dimensions and head values shown of the infiltrometer used in Figure 3.1c.

3.6 CONCLUSIONS

In this study factors such as capping condition and H:D ratio were evaluated for their effects on laboratory compressive strength measurements of pervious concrete. Several mix designs were tested with all specimens 4” (10.1 cm) in diameter and prepared using a 3/8” (0.9 cm) aggregate. Specimens with H:D ratios of 2:1, 1.5:1 and 1:1 were tested for compressive strength with 2:1 and 1.5:1 specimens capped using rubber capping and sulfur capping methods. Relations between field infiltration testing methods (single ring, double ring and falling head infiltrometers) and laboratory hydraulic conductivity measurements on pervious concrete were also investigated. Two field sites and laboratory slab samples of previous concrete were tested with the three infiltration methods and the hydraulic conductivity testing was performed on cylindrical specimens cored from the slab specimens. Based on the results of this investigation the following conclusions are drawn. The conclusions are generally valid for pervious concrete made using 3/8” (0.95 cm) aggregate. In addition the conclusions related to infiltration testing are valid for 6” (15.2 cm) slabs and for the falling head infiltrometer dimensions and head values used in this study. PCP thicknesses different than the 6” (15.2 cm) evaluated in this study may result in different correlations between the infiltration methods investigated in this study. However the correlations presented can be used for other pavement thicknesses as a rough estimate when needed.

- Both methods of end capping were found to provide comparable compressive strength measurements; however, sulfur capping yielded measurements with greater variability. For this reason, rubber capping is recommended over sulfur capping. The rubber capping procedure is also simpler compared to the sulfur capping procedure.
- Using a H:D ratio smaller than the recommended 2:1 ratio increased compressive strength measurements by about 10%. In comparison to H:D of 2:1 and 1.5:1, the specimens with H:D of 1:1 yielded relatively inconsistent compressive strength measurements. It is suggested that if H:D of less than 2:1 is used for compression strength testing with rubber capping, the measured strength may be reduced by 10% to estimate the strength with H:D of 2:1. If possible a H:D ratio smaller than 1.5:1 should be avoided.
- The infiltration testing methods evaluated in this study were found to have linear correlations to one another. The infiltration rates from single ring, double ring and falling head infiltrometers correlated as 1: 0.85: 4.7. The double ring infiltrometer provided meaningful measurements for infiltration rates less than about 150 in/hr (0.10 cm/sec). This was because at greater infiltration rates it is difficult to keep a constant water level in the outer ring.
- The hydraulic conductivity correlated linearly with the infiltration rates from the single ring and falling head infiltrometers as 1: 1.8: 9.0. These correlations can be used to estimate hydraulic conductivity from field measurements of infiltration rates.
- The infiltration rate measured using the falling head infiltrometer correlated very well with that from the single ring infiltrometer. The falling head infiltrometer is easier to use

because there is no need for sealants, it requires less water, and it is simple to operate; and therefore, may be used in place of the single ring infiltrometer, especially in comparative studies (e.g. long term monitoring). If desired, the single ring infiltrometer-based infiltration rate can be estimated by dividing the falling head infiltrometer measurement by 4.9 (~5) provided that the same dimensions and head values as adopted in this research are used.

3.7 REFERENCES

ACI Committee 522 (2010). “Report on Pervious Concrete 522R-10,” American Concrete Institute, Farmington Hills, Mich., 2010.

ASTM C39 (2010). “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C192 (2010). “Standard Practice for Making and Curing Concrete Test Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C617 (2010). “Standard Practice for Capping Cylindrical Concrete Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C1231 (2010). “Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C1701 (2010). “Standard Test Method for Infiltration Rate of In Place Pervious Concrete.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Bean, E. Z., Hunt, W. F., and Bidelspach, D. A. (2007). “Field Survey of Permeable Pavement Surface Infiltration Rates.” J. of Irrigation and Drainage Engineering. Vol. 133, No. 3, pp. 249-255.

Berry B., Anderson I., Suozzo M., Dewoolkar M.M. (2012), "Properties of Pervious Concrete Incorporating Recycled Concrete Aggregate", Accepted as part of the Transportation Research Board Annual Meeting. January, 2012. Washington D.C.

Chindaprasirt P., Hatanaka S., Chareerat T., Mishima N. and Yuasa Y.(2008). “Cement Paste Characteristics and Porous Concrete Properties.” Construction and Building Materials, Volume 22, Issue 5, May 2008, pp. 894-901.

Chopra, M., Kakuturu, S., Ballock, C. Spence, J. (2010). “Effect of Rejuvenation Methods on the Infiltration Rates of Pervious Concrete Pavements” Journal of Hydraulic Engineering, Vol. 15, No. 6, June 2010.

Chopra, M. and Wanielista, M. (2007). “Performance Assessment of Portland Cement Pervious Pavement: Compressive Strength of Pervious Concrete Pavements.” Stormwater Management Academy, University of Central Florida.

Ferguson, Bruce K. (2005), Porous Pavements, Boca Raton, FL: CRC Press.

Ghafoori, N., and Dutta, S. (1995a). “Laboratory Investigation of Compacted No-Fines Concrete for Paving Materials.” Journal of Materials in Civil Engineering, Vol. 7, No. 3, August 1995, pp.

183-191.

Head W.J., Iskander, W.H., Folden, J.D. (1999). “Assessment of Steel End Caps with Rubber Inserts as Capping Media for Concrete Cylinders.” Final Report WVDOH RP# 136. West Virginia University.

Henderson, V., Tighe, S.L., and Norris, J. (2009). “Pervious Concrete Pavement: Integrated Laboratory and Field Study.” Transportation Research Record: Journal of the Transportation Research Board. No. 2113, Washington, DC, pp. 13-21.

Klieger P. and Lamond J. F. (1994). “Significance of Tests and Properties of Concrete and Concrete-Making Materials” American Standard Testing Methods, Fredericksburg, VA.

McCain, G. N., and Dewoolkar, M. M. (2010), "Porous concrete pavements: mechanical and hydraulic properties", Transportation Research Record, No. 2164,Washington D.C., 66-75.

Meininger, R.C., “No-Fines Pervious Concrete for Paving,” Concrete International, Vol. 10, No. 8, August 1988, pp. 20-27.

Montes, F. and Haselbach, L.M. (2006). “Measuring Hydraulic Conductivity in Pervious Concrete.” Environmental Engineering Science. Nov 2006, Vol. 23, No. 6, pp. 960-969.

National Ready Mixed Concrete Association (NRMCA) (2004). “Freeze-Thaw Resistance of Pervious Concrete.” Silver Spring, MD.

Rizvi, R., Tighe, S., Henderson, V., and Norris, J. (2009). “Laboratory sample preparation techniques for Pervious Concrete.” TRB Annual Meeting. Report No. 09-1962.

Sansalone, J., Kuang, X., and Ranieri, V. (2008). “Permeable Pavement as a Hydraulic and Filtration Interface for Urban Drainage.” Journal of Irrigation and Drainage Engineering Volume: 134. Issue: 5. Pages: pp. 666-674.

Schaefer, V., Wang, K., Suleimman, M. and Kevern, J. (2006). “Mix Design Development for Pervious Concrete in Cold Weather Climates.” Final Report, Civil Engineering, Iowa State University.

Tennis, P. D., Leming, M. L., and Kiefer, C. J. (2004). “Pervious Concrete Pavements.” Portland Cement Association, Skokie, IL.

Table 0.1 - Pervious concrete mix designs used in this study

Mix Number	Cement lbs/yd3 (kg/m3)	Aggregate lbs/yd3 (kg/m3)	Water lbs/yd3 (kg/m3)	AEA oz/yd3 (ml/m3)	HRWR oz/yd3 (ml/m3)	VMA oz/yd3 (ml/m3)	Stabilizer oz/yd3 (ml/m3)	Fly Ash lbs/yd3 (kg/m3)	RCA lbs/yd3 (kg/m3)
RAND-1	630 (373)	2,798 (1,660)	158 (93)	3 (67)	17 (384)	40 (904)	40 (904)	-	-
RAND-2	630 (373)	2,798 (1,660)	184 (109)	3 (67)	17 (384)	40 (904)	40 (904)	-	-
RAND-3	630 (373)	2,798 (1,660)	209 (124)	3 (67)	17 (384)	40 (904)	40 (904)	-	-
HERT-1	600 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	-
HERT-2	600 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	30 (17)	-
HERT-3	600 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	60 (35)	-
HERT-4	600 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	120 (71)	-
RCA-1	601 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	-
RCA-2	602 (355)	2,343 (1,390)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	257 (152)
RCA-3	603 (355)	2,077 (1,232)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	523 (310)
RCA-4	604 (355)	1,820 (1,079)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	780 (462)
RCA-5	605 (355)	1,297 (769)	165 (97)	1 (22)	18 (406)	12 (271)	-	-	1,303 (773)
RCA-6	606 (355)	-	165 (97)	1 (22)	18 (406)	12 (271)	-	-	2,600 (1,542)
FIELD	607 (355)	2,600 (1,542)	165 (97)	1 (22)	18 (406)	12 (271)	-	30 (17)	-

Table 0.2 - Summary of results of rubber capped and sulfur capped testing

		Sulfur Capped			Rubber Capped		
		Unit Weight	Compressive Strength		Unit Weight	Compressive Strength	
Mix Design	H:D Ratio	Average lbs/ft3 (kg/m3)	Average psi (MPa)	Standard Deviation psi (MPa)	Average lbs/ft3 (kg/m3)	Average psi (MPa)	Standard Deviation psi (MPa)
RCA-1	1.5:1	115.9 (1,858)	2,252 (15.53)	497 (3.43)	115.3 (1,847)	2,580 (17.79)	239 (1.65)
RCA-2	1.5:1	117.5 (1,882)	2,272 (15.67)	259 (1.78)	116.9 (1,873)	2,438 (16.81)	135 (0.93)
RCA-3	1.5:1	119.2 (1,910)	2,766 (19.07)	461 (3.18)	118.8 (1,904)	2,885 (19.89)	221 (1.52)
RCA-4	1.5:1	116.7 (1,868)	2,024 (13.95)	113 (0.78)	115.8 (1,855)	2,252 (15.52)	105 (0.73)
RCA-5	1.5:1	116.5 (1,865)	2,161 (14.89)	292 (2.01)	115.2 (1,846)	2,530 (17.44)	195 (1.34)
RCA-6	1.5:1	112.4 (1,800)	2,116 (10.28)	430 (2.96)	111.7 (1,790)	1,994 (13.74)	95 (0.66)
HERT-1	2:1	111.6 (1,787)	1,491 (10.28)	153 (1.05)	112.6 (1,804)	1,422 (9.80)	197 (1.36)
HERT-2	2:1	113.1 (1,812)	1,652 (11.39)	269 (1.85)	114.0 (1,826)	1,530 (10.55)	115 (0.79)
HERT-3	2:1	112.4 (1,801)	1,663 (11.47)	303 (2.08)	114.3 (1,831)	1,519 (10.47)	169 (1.16)
HERT-4	2:1	107.9 (1,728)	1,000 (6.90)	261 (1.80)	108.6 (1,739)	923 (6.36)	139 (0.96)

Table 0.3 - Summary of results of height to diameter testing

	1:1 H:D Ratio Specimens			1.5:1 H:D Ratio Specimens			2:1 H:D Ratio Specimens		
	Unit Weight	Compressive Strength		Unit Weight	Compressive Strength		Unit Weight	Compressive Strength	
Mix Design	Average lbs/ft3 (kg/m3)	Average psi (MPa)	Standard Deviation psi (MPa)	Average lbs/ft3 (kg/m3)	Average psi (MPa)	Standard Deviation psi (MPa)	Average lbs/ft3 (kg/m3)	Average psi (MPa)	Standard Deviation psi (MPa)
RCA-1	-	-	-	115.3 (1,847)	2,580 (17.79)	239 (1.65)	116.8 (1,871)	2,161 (14.90)	250 (1.72)
RCA-2	-	-	-	116.9 (1,873)	2,438 (16.81)	135 (0.9)	116.1 (1,860)	2,246 (15.48)	226 (1.56)
RCA-3	-	-	-	118.8 (1,904)	2,885 (19.89)	221 (1.52)	117.2 (1,878)	2,57 (17.72)1	197 (1.36)
RCA-4	-	-	-	115.8 (1,855)	2,252 (15.52)	105 (0.73)	113.6 (1,820)	2,024 (13.95)	83 (0.57)
RCA-5	-	-	-	115.2 (1,846)	2,530 (17.44)	195 (1.34)	114.1 (1,829)	2,051 (14.14)	188 (1.29)
RCA-6	-	-	-	111.7 (1,790)	1,994 (13.74)	95 (0.66)	111.6 (1,787)	1,815 (12.51)	172 (1.18)
HERT-1	108.9 (1,745)	1,859 (12.82)	225 (1.55)	111.8 (1,791)	1,514 (10.44)	240 (1.66)	112.6 (1,804)	1,422 (9.80)	197 (1.36)
HERT-2	110.0 (1,763)	2,014 (13.88)	259 (1.78)	113.2 (1,813)	1,864 (12.85)	161 (1.11)	114.0 (1,826)	1,530 (10.55)	115 (0.79)
HERT-3	110.8 (1,775)	2,118 (14.60)	199 (1.37)	112.7 (1,806)	1,705 (11.75)	147 (1.01)	114.3 (1,831)	1,519 (10.47)	169 (1.16)
HERT-4	104.2 (1,670)	1,087 (7.49)	217 (1.50)	105.2 (1,686)	1,017 (7.01)	167 (1.15)	108.6 (1,739)	923 (6.36)	139 (0.96)
RAND-1	109.9 (1,761)	465 (3.21)	193 (1.33)	116.9 (1,873)	844 (5.82)	138 (0.95)	114.6 (1,836)	526 (3.63)	138 (0.95)
RAND-2	120.3 (1,927)	1,622 (11.18)	569 (3.92)	125.3 (2,008)	1,962 (13.53)	387 (2.67)	121.8 (1,952)	1,568 (10.81)	382 (2.63)
RAND-3	127.9 (2,049)	2,897 (19.97)	509 (3.51)	132.7 (2,127)	2,817 (19.42)	400 (2.76)	130.2 (2,086)	3,011 (20.76)	305 (2.10)

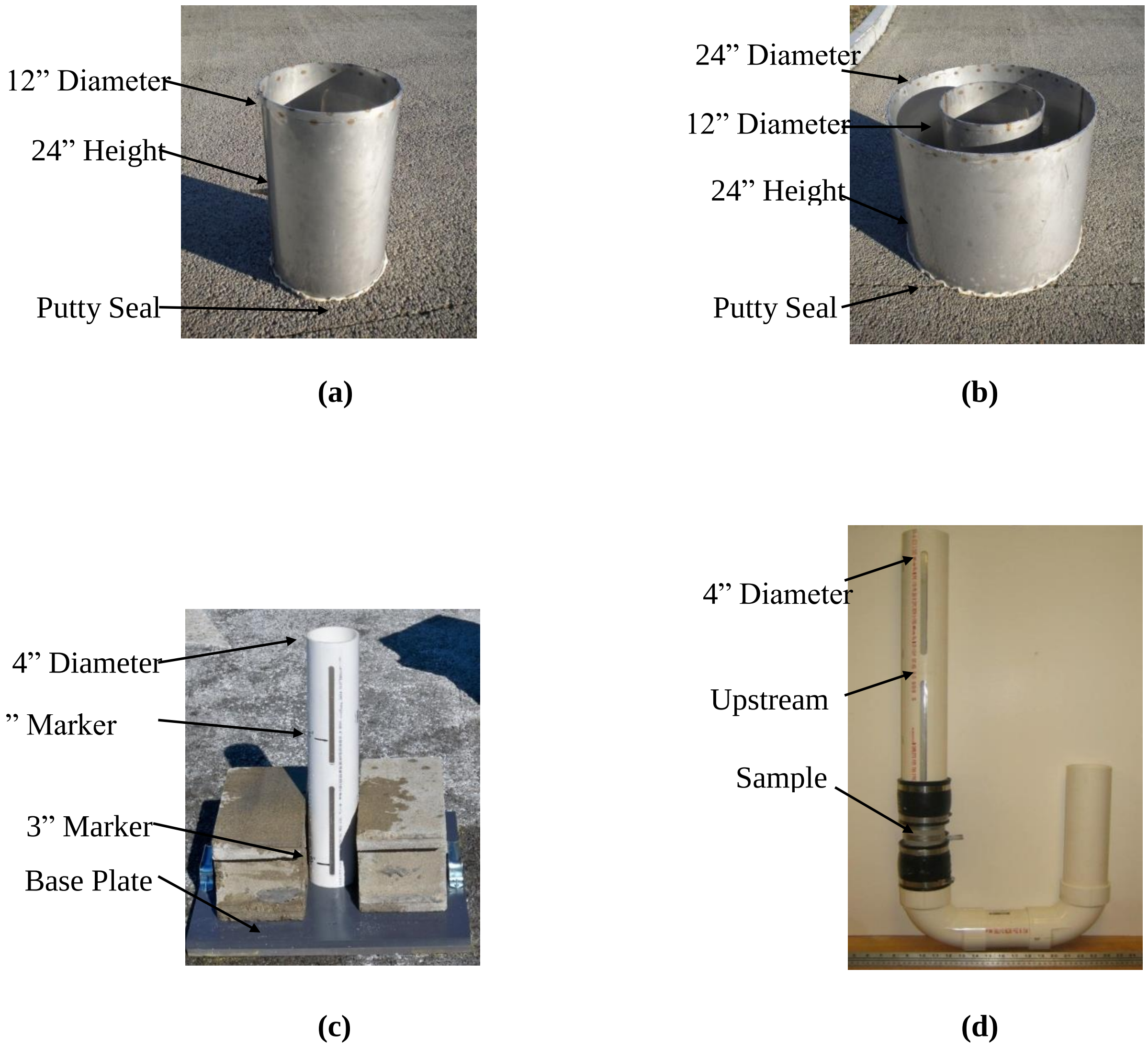
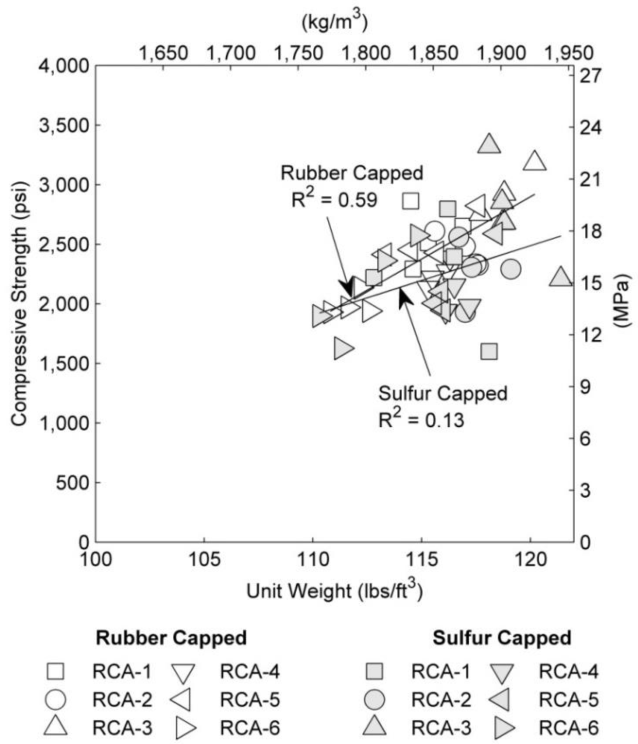
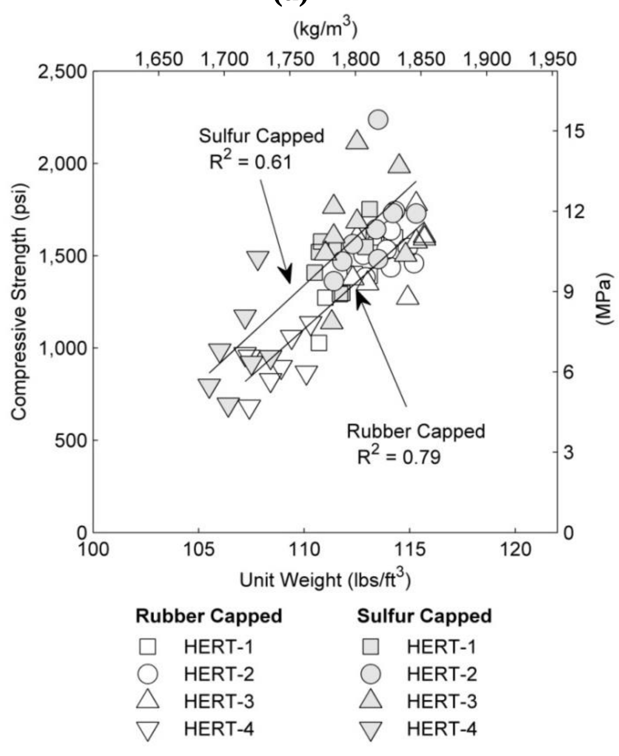


Figure 0.1 - (a) Single Ring Infiltrometer, (b) Double Ring Infiltrometer, (c) Falling Head Infiltrometer and (d) Falling Head Permeameter.



(a)



(b)

Figure 0.2 - Effects of capping on (a) RCA mix designs, 6” (15.2 cm) specimens and (b) HERT mix designs 8” (20.3 cm) specimens.

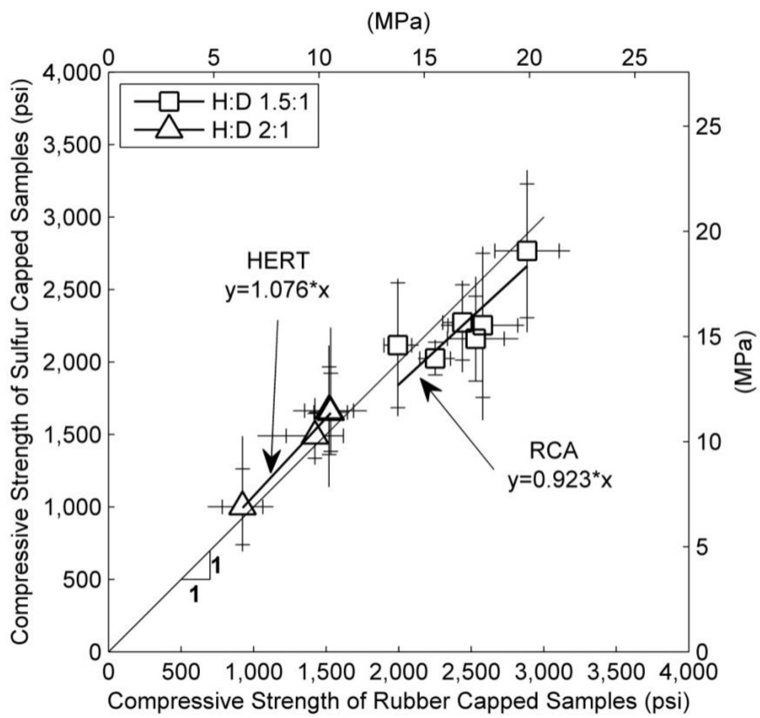
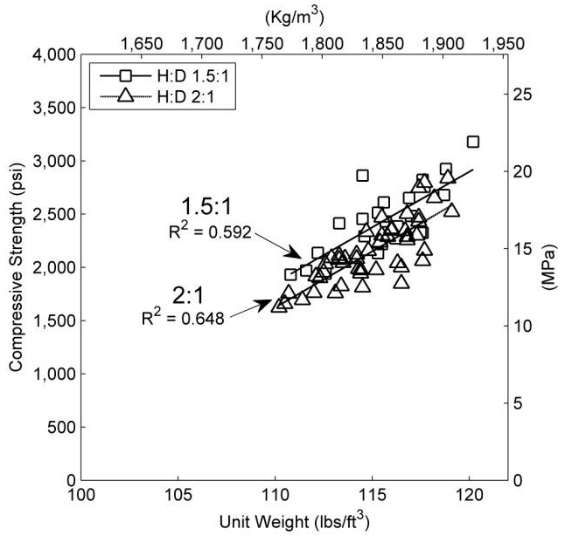
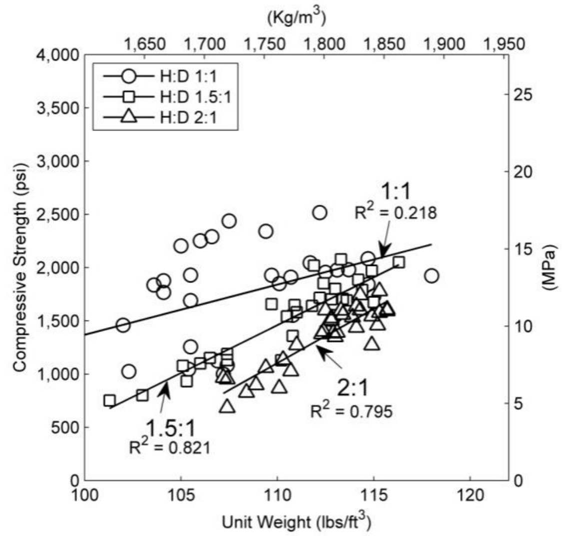


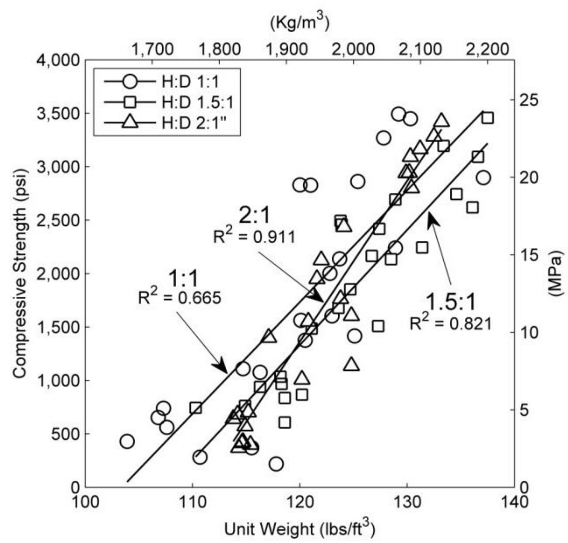
Figure 0.3 - Average compressive strength as a function of capping condition.



(a)

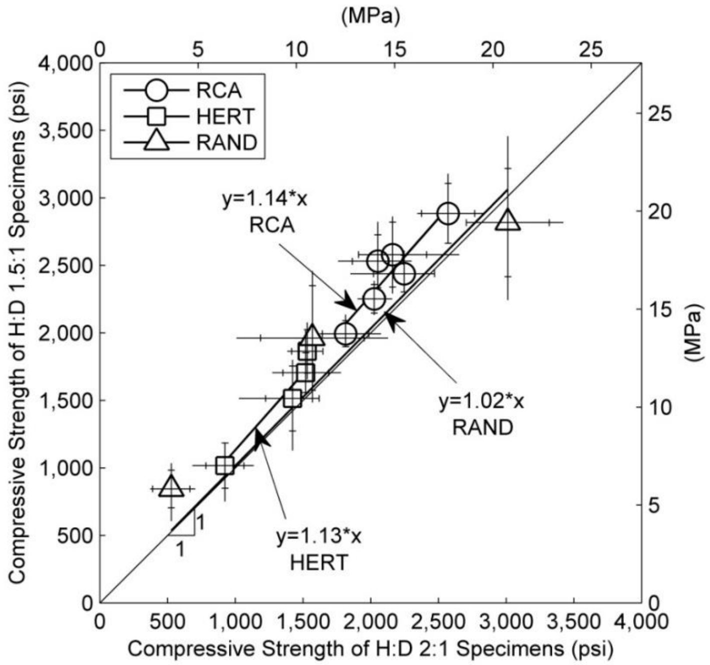


(b)

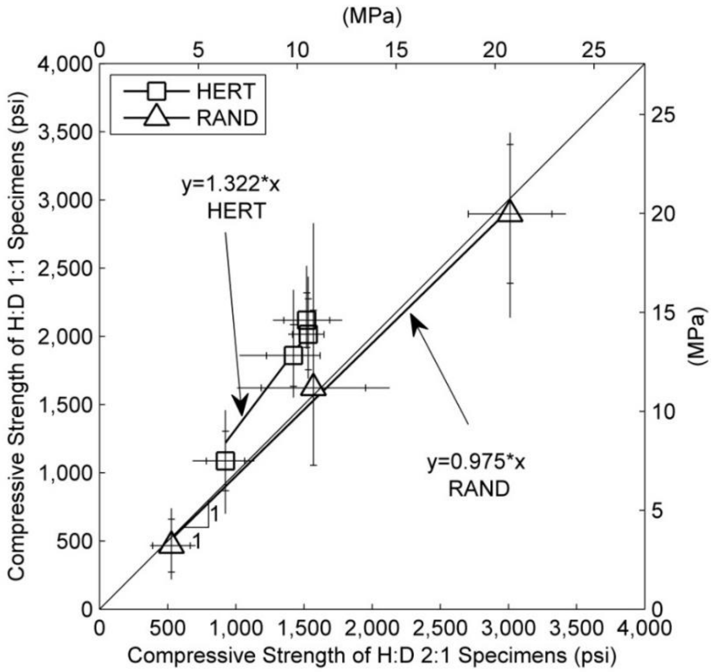


(c)

Figure 0.4 - Effect of height to diameter ratio for (a) RCA mixes, (b) HERT mixes, and (c) RAND mixes.



(a)



(b)

Figure 0.5 - Results of compressive strength testing for (a) 2:1 and 1.5:1 H:D specimens, and (b) 2:1 and 1:1 H:D specimens.

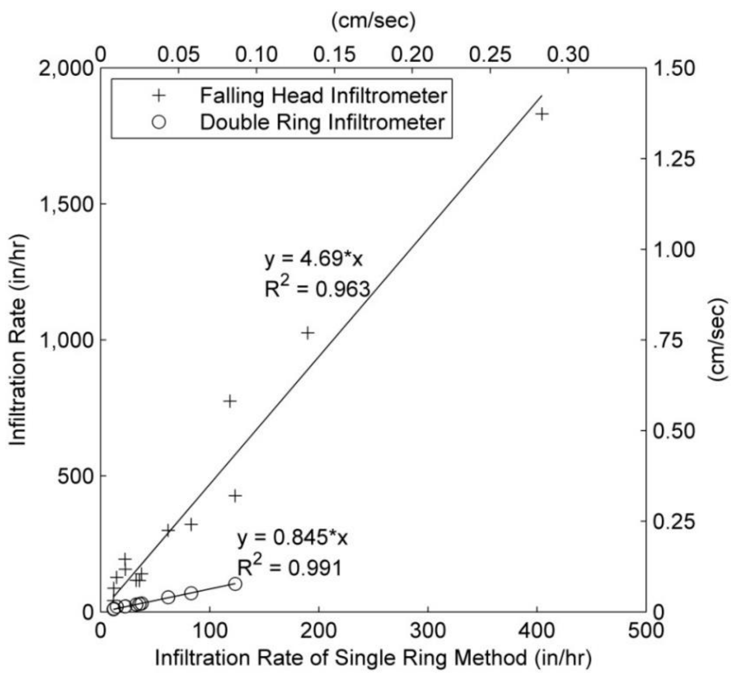


Figure 0.6 - Results of field measurements of infiltration rates.

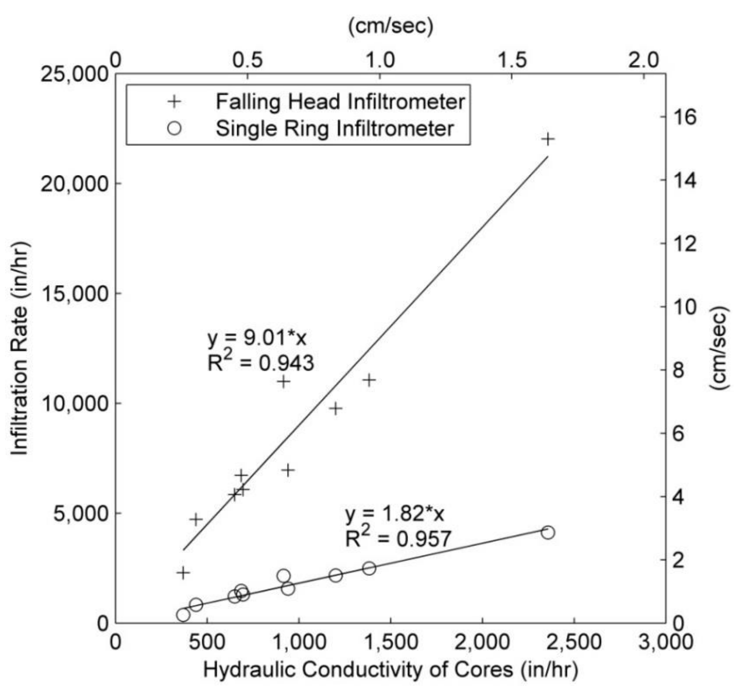


Figure 0.7 - Results of laboratory measurements of infiltration rate compared to results of hydraulic conductivity measurements of cores.

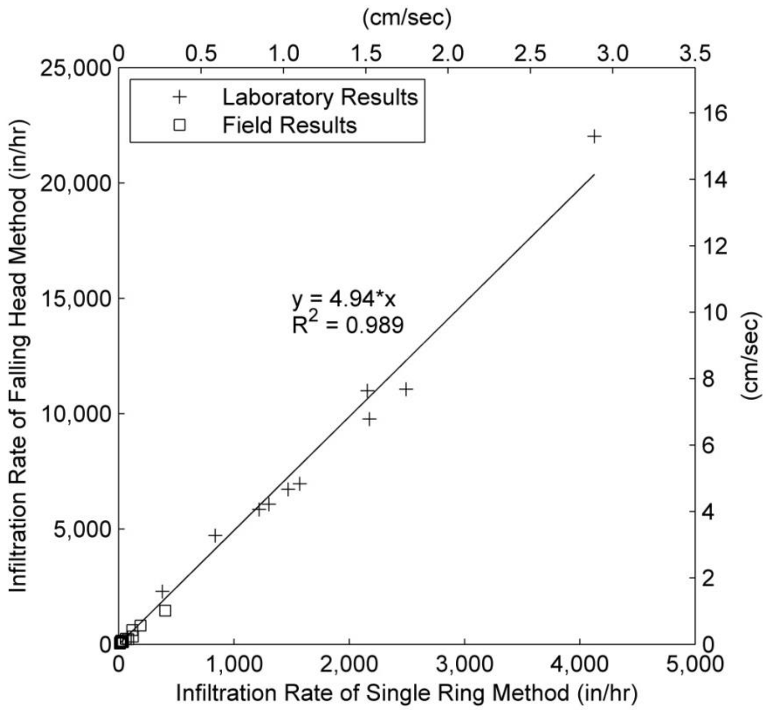


Figure 0.8 - Comparison of falling head infiltrometer results to single ring infiltrometer results.

CHAPTER 4

LONG-TERM FIELD MONITORING AND EVALUATION OF MAINTENANCE PRACTICES OF PERVIOUS CONCRETE PAVEMENTS IN VERMONT

4.1 ABSTRACT

Pervious concrete pavement (PCP) is used due to its unique properties allowing water to infiltrate into the surface. The objectives of this study were to observe the performance of PCP in the field; determine the effectiveness of cleaning methods to restore infiltration rates; and compare field observations to available laboratory results when possible. Two PCP sites in Vermont were monitored over a year long period by measuring infiltration rates at several locations. Facility-wide cleaning operations such as street sweeping and vacuum truck cleaning were tested for their ability to restore infiltration rates along with spot cleaning methods including hand vacuuming, pressure washing and a combined method. Infiltration rates decreased gradually during the monitoring period with average reductions of 59% at the first facility and 26% at the second facility. Street sweeping and vacuum truck cleaning were found to restore infiltration rates by 21% and 30% respectively, but were not able to restore severely clogged areas. Spot cleaning methods increased infiltration rates by 85% after pressure washing, 10% after vacuuming and 100% after pressure washing followed by vacuuming. Vacuum truck cleaning is recommended; however, either method should be used for maintenance operations starting after construction. Spot cleaning methods with the exception of vacuuming were found to restore infiltration rates of severely clogged areas and are recommended for localized cleaning. Results of long-term monitoring compared reasonably well with previous studies; whereas the results associated with cleaning were substantially lower compared to results found in the literature.

4.2 INTRODUCTION

4.2.1 Research Motivation

Pervious concrete pavements (PCP) have been identified by federal and state agencies as a management practice for the treatment of stormwater by reducing the volume of runoff from impervious surfaces (ACI, 2010). PCP incorporates an open pore structure allowing water to travel from the surface into a gravel subbase and subsequently into the native soil. PCP has additional benefits including, reducing noise from vehicles, improving skid resistance and reducing heat island effects (Olek et al., 2003; Tennis et al., 2004; and PCA, 2003). Due to the open pore structure, PCP is generally limited to areas where lower strength is acceptable such as parking lots, low volume roads, driveways and sidewalks.

Ensuring the long-term performance of PCP is a major concern to designers and maintenance personnel. PCP can be compromised in two ways; clogging of the pores reducing infiltration capacity or damage to the structure of the PCP leading to failure of the pavement. The open pore structure of PCP gradually accumulates inorganic and organic materials leading to reduced infiltration rates. This reduction could lead to the PCP becoming an impervious surface unless periodic facility-wide maintenance such as street sweeping or vacuum truck cleaning is performed or hand-held spot cleaning methods such as hand vacuuming or pressure washing are performed (ACI, 2010). Several researchers have attempted to model the effects of clogging and cleaning on PCP in the laboratory; however, there are only a few field studies available to compare with laboratory results.

In order to ensure the long-term performance of PCP, their maintenance practices must be understood. The purpose of this study is to evaluate the effects of typical maintenance practices used on PCP by conducting field investigations and comparing the results to laboratory studies. The specific objectives of this study are to (i) monitor infiltration rate of two PCP facilities in Vermont; (ii) determine the effectiveness of various methods to restore infiltration rate in the field; and (iii) compare field observations to previous laboratory studies.

4.3 BACKGROUND

This section presents a literature review related to the effects of maintenance activities on PCP. Studies related to clogging and cleaning of PCP in the field and in the laboratory are also presented.

4.3.1 Field Infiltration Measurements and Restoration

The field infiltration rate of various pervious pavement surfaces including PCP have been investigated using single and double ring infiltrometers modified from soils testing (Bean et al., 2007). Results from various PCP facilities showed infiltration rates ranging from 5 to 2,750 in/hr (0.003 to 1.91 cm/sec). The authors noted that areas with visible signs of clogging had much lower infiltration rates when compared to areas that showed no signs of clogging. Based on these results the authors recommended that regular cleaning be performed to prevent clogging.

Henderson et al. (2009) measured infiltration rates using a Gibson asphalt permeameter at sixteen locations at each of the three PCP study sites in Canada during summer 2007, winter 2008 and summer 2008. Sites were classified into areas receiving sand as a winter maintenance practice and areas not receiving sand. The infiltration rate decreased over time at two of the tested locations. Based on the results it was determined that sanding did not increase the clogging of PCP compared to areas not sanded. Sites were cleaned with a 6-hp wet-dry vacuum to restore infiltration rate at one facility. Pre-vacuuming infiltration rates ranged from 4.2 to 150 in/hr (0.003 to 0.106 cm/sec) while post-vacuuming rates ranged from 190 to 5,200 in/hr (0.139 to 3.72 cm/sec) increasing infiltration rates by a factor of 1.3 to 287. Based on these results the researchers recommended vacuuming as a rehabilitation method.

Chopra et al. (2010) conducted field and laboratory investigations into various methods to restore infiltration rates of PCP. Eight facilities were selected in southeastern United States with infiltration rates measured in the field using a modified single ring infiltrometer. This infiltrometer penetrated both the PCP and the subsoil, reducing lateral flow during testing. The infiltration rates of the PCP and subsoil were measured; the PCP section was then removed and measured for hydraulic conductivity in the laboratory. Based on the results of the field and laboratory testing the authors were able to determine if the PCP or subsoil was the limiting factor for infiltration. The results showed several PCP cores were limiting sections. These cores were cleaned using vacuuming, pressure washing or a combination of both methods, and tested for hydraulic conductivity before and after cleaning to determine treatment efficiencies. Results indicated that hydraulic conductivity values could be increased by a factor of 10 by vacuuming, 56 by pressure washing and 66 by combining the two methods. The authors concluded that these rejuvenation methods should be useful in the field.

4.3.2 Laboratory Studies

Montes and Haselbach (2006) covered PCP specimens with fine sand of known permeability and simulated various rain events and slopes in a flume. Infiltration rates decreased significantly during testing, with pre-clogging values ranging from 280 to 1,400 in/hr (0.2 to 1.0 cm/sec) and post-clogging values ranging from 3 to 7 in/hr (0.002 to 0.005 cm/sec). The researchers concluded that the observed infiltration rates were sufficient to handle the expected 100-year storm event.

Joung and Grasley (2008) investigated the hydraulic conductivity of clean and clogged PCP specimens with a falling head permeameter. Clogging was accomplished by passing a mixture of sand and water through the specimen multiple times. After water had drained from the specimen, hydraulic conductivity tests were performed a second time to measure the reductions. The results showed the sand did not clog PCP specimens with a void ratio of 33% or greater. Specimens with a lower void ratio were affected, with the hydraulic conductivity reduced by approximately 40%.

Deo et al. (2010) investigated the clogging of PCP for varying aggregate size by measuring hydraulic conductivity using a falling head permeameter. Clogging was accomplished by adding 25g of sand to the top of the specimen and performing additional permeability testing that allowed water to transport sand into the PCP specimen. This process was repeated until additional sand application did not result in further reductions in infiltration rate. They observed 30% reductions in hydraulic conductivity after clogging.

4.4 RESEARCH METHODS

Field investigations were conducted to determine the following; (i) how infiltration rate of PCP changes over time, (ii) how facility-wide cleaning operations such as street sweeping and vacuum truck cleaning restore infiltration rates, and (iii) how spot cleaning methods such as hand vacuuming and pressure washing restore infiltration rate of PCP in severely clogged locations.

4.4.1 Surface Infiltration Capacity

A falling head infiltrometer was designed and built at the University of Vermont to monitor the long-term infiltration rate of two PCP facilities. A sheet of PVC measuring 2' x 2' x ¾ in (60.9 cm x 60.9 cm x 1.90 cm) with a circular opening for a standpipe was used as a base. The standpipe was made with PVC pipe 0.25 in (0.63 cm) thick and with an internal diameter of 4 in (10.1 cm). Milled viewports on the standpipe allowed for monitoring of water levels during testing. A foam rubber ring was attached to the underside of the device around the standpipe to create a seal between the infiltrometer and the PCP surface. Weights (about 120 lbs [55 kg]) were placed onto the device to compress the foam during testing as seen in.

The infiltration rate was determined by filling the standpipe and measuring the time for the water level to drop from 15 in (38.1 cm) to 3 in (7.6 cm) above the PCP. Previous research on PCP indicated laminar conditions would exist for the head values used during testing (Montes and Haselbach, 2006). Measurements were taken three times at each location to ensure consistent results. Infiltration rate was determined using Eq. 1 below. This falling head infiltrometer method has been found to correlate well with the ASTM standard (single ring) for infiltration measurements of PCP and to saturated hydraulic conductivity measurements with a relation of 1 : 1.8 : 9 (hydraulic conductivity : ASTM standard : falling head infiltrometer) (Suozzo and Dewoolkar, 2011) .

$$I = \frac{c\left(h_1 - h_2\right)}{t} \tag{1}$$

where,

I = Infiltration rate, (in/hr),
t = Recorded time, (s),
h_1 = Initial water level, (in),
h_2 = Finial water level, (in), and
c = Conversion factor = 3,600 s/hr.

4.4.2 Field Facilities

Infiltration testing was performed at two PCP facilities. The first facility is located at College Street in Burlington, Vermont, and was constructed in June 2009. PCP is used in the central portion of the parking area, with the remainder traditional asphalt. Figure 4.2 shows an outline of the site with infiltration monitoring sites listed alphabetically and cleaning locations listed numerically. The site is graded to direct water to the central PCP section. Below the pavement surface is a 34-inch layer of gravel used as storage reservoir and perforated pipes that remove the collected stormwater. No cleaning operations were performed on the facility prior to this investigation. Winter maintenance consisted of plowing with little use of sand or salt; however, both were used on a nearby road with runoff from the road entering the facility.

The second facility is located at Heritage Flight in South Burlington, Vermont, and was constructed in September 2009. The lot is solely PCP. An outline of the facility is shown in Figure 4.3, with infiltration monitoring sites and cleaning sites shown as in Figure 4.2. The area is primarily a parking facility; however, a small amount of truck travel occurs due to fuel deliveries. The facility is graded flat with minimal runoff from the adjacent impervious surfaces. The PCP overlays a gravel base 32-inches (81.2 cm) thick with water infiltrating into native soils. The facility was cleaned by street sweeping bi-annually, once in the fall after foliage had dropped and once in the spring after winter maintenance ended. Winter maintenance consisted of plowing with a rubber tipped blade with no application of sand or salt.

4.4.3 Long-Term Field Monitoring

The infiltration rate was measured at eight locations at the College Street facility, with sites A, C, F and H located near the PCP/asphalt divide and sites B, D E and G located near the inner island. Fourteen locations were selected at the Heritage lot, with sites at areas: (i) outside the wheel path experiencing little traffic (A, C, N, and P), (ii) inside the wheel path experiencing traffic (B, D, O, and Q), (iii) in parking areas not under tires (E, G, and I), and (iv) in parking areas under tires (F, H, and K). Infiltration measurements were in August 2010 at both locations, with subsequent measurements taken every month. Although infiltration data immediately after construction is not available, infiltration rates of slabs created in the laboratory using an identical mix design and similar placement methods were used to estimate a range of post-construction infiltration rates.

4.4.4 Infiltration Recovery Methods

Cleaning operations were separated into two categories: (i) facility wide cleaning operations; which included street sweeping and vacuum truck cleaning; and (ii) spot cleaning methods such as vacuuming by hand, pressure washing and a combination of the two methods. Street sweeper cleaning was performed using an Elgin Whirlwind air sweeper that brushed material from pores and carried it to a central vacuum. This vacuum removed material; however, it did not form a tight seal with the pavement surface. Vacuum cleaning was performed using a Tymco 500x regenerative air vacuum truck. This system utilized a vacuum system providing a better seal to the pavement surface and included pressurized air to dislodge material from the pores and remove it from the surface. Street sweeping was performed as part of regular maintenance at the Heritage facility in September of 2010 and vacuum truck cleaning was conducted at both facilities in October 2011. To determine the effectiveness of these maintenance operations infiltration rates were measured before and after cleaning at eight locations per facility. Handheld methods were tested to determine their effectiveness at restoring infiltration at four locations at the College Street facility and ten locations at the Heritage facility. Sites selected for vacuuming were cleaned by a 5-hp wet-dry vacuum; an area measuring 2’x2’ (61cm x 61cm) was vacuumed for 30 seconds in one direction, then another 30 seconds in the perpendicular direction. Locations selected for pressure washing were cleaned with a 3,500-psi pressure washer; the method of cleaning was identical to the vacuuming procedure. The combined method was used at sites by first pressure washing then vacuuming. Pre and post cleaning infiltration rates for all methods were determined using the falling head infiltrometer.

4.5 RESULTS AND DISCUSSION

4.5.1 Long-Term Field Monitoring

Results of long-term monitoring of the College Street facility are presented in Figure 4.4. Figure 4.4a shows monitoring sites along with the infiltration rate (in/hr) at the beginning and end of monitoring period (August 2010 to July 2011), and the percent decrease. Infiltration values reported as CLG indicated complete clogging. Infiltration rate likely ranged from 1,400 to 2,800 in/hr (1.0 to 2.0 cm/sec) immediately after construction.

Initial infiltration rate of sites near the inner island are significantly lower than other locations, with sites B, D, and G having the lowest infiltration rates at the start of testing. The initial infiltration of other sites was higher, ranging from 574 to 1,442 in/hr (0.41 to 1.03 cm/sec). Figure 4.4b shows areas where clogging from sand or organic matter was visible at the end of the monitoring period; clogging was identified by standing water after storm events. Clogging primarily occurred at locations near the inner island and was present on both sides of the facility. Clogging near site G consisted mostly of sand, whereas near sites D and B clogging consisted of a combination of organic matter and sand. Visibly clogged areas covered approximately 15% of the facility. Figure 4.4c shows locations that appeared to be raveling during the monitoring period, these locations closely matched areas of severe clogging. These observations support previous laboratory studies indicating that clogging can accelerate freeze-thaw damage in PCP (Guthrie et al., 2010). Due to the high initial clogging, these sites do not show significant change over the testing period. Other locations showed decreases in infiltration rates of 42% to 64%. Average infiltration values determined at each location are presented in Figure 4.4d. Infiltration measurements were not recorded in December and May due to poor weather conditions. Infiltration rates generally decreased continually throughout the monitoring period with infiltration rates at sites A, C, and E decreasing by the largest amount. This gradual decrease suggests that clogging material is constantly being transported to the site with little variation over the year long period.

Results of long-term monitoring at the Heritage facility are presented in Figure 4.5. Clogging at the beginning of monitoring was observed at sites B, and D as seen in Figure 4.5a. Delivery trucks, that can compact sand and other materials into pores, regularly travel these sites. Infiltration rates at sites in a similar location but not experiencing truck traffic (A and C) were an order of magnitude higher. This difference was not as drastic in the sites with a similar distribution inside and outside of the travel lane, but did not regularly experience truck traffic (N though Q), indicating that not only vehicle traffic but the type of vehicle can impact the clogging of PCP. Sites located in the parking areas showed the highest initial and final infiltration values, with little difference between sites inside or outside the travel lane. Figure 4.5b shows visible clogging located near entrances to the facility. Clogging material was found to consist mostly of asphalt particles from the nearby road. Raveling, shown in Figure 4.5c, was not correlated with clogging at the Heritage facility and was observed to be more random in location. Both clogging and raveling were observed on less than 5% of the surface. As with the College Street facility, unclogged areas showed similar reductions to infiltration rate; most sites decreased 10% to 30% over the testing period. Figure 4.5d and e show the range of infiltration rates observed in travel lanes and parking areas respectively. All sites with the exception of C showed a gradual decrease over time with infiltration values remaining high at the end of testing. Differences existed between infiltration rates of sites located within the wheel track and sites outside the wheel track as seen in Figure 4.5d. Several sites presented in Figure 4.5e show infiltration rates above the range of initial infiltration values. These results are likely due to variability of the construction, specifically low compaction resulting in higher infiltration rates. In the parking areas there was little difference between the sites inside the wheel track and sites outside the wheel track.

4.5.2 Infiltration Recovery Methods

4.5.2.1 Facility Wide Cleaning

The effects of facility wide maintenance practices are presented in Figure 4.6. Bars show the initial infiltration value measured at the beginning of the monitoring period with measurement before and after cleaning by both street sweeper and vacuum truck. For each cleaning method the absolute increase to infiltration rate (in/hr) and percentage increase are presented above the bar plots. A range of estimated post construction infiltration values and the infiltration rate associated with the 100-year, 24-hour storm event for the region are shown. Cleaning by sweeper was only performed at the Heritage facility and is shown in Figure 4.6a. Sweeping primarily collected dislodged aggregate and cement paste along with large inorganic and organic material on the PCP surface. The average increase to infiltration values was 28% due to cleaning. However, this value includes results from clogged sites that showed large increases to infiltration rate due to low pre cleaning infiltration rates. These sites do not accurately represent the average conditions of the facility and inflate the average recovery value. Removing these sites (B and D) from the analysis results in an average increase to infiltration rate of 21%, which more accurately represents the effect on non-clogged areas. During post cleaning observations material was still present in the pore structure indicating that cleaning was incomplete.

Vacuum truck cleaning was performed on both sites with differences in effectiveness. At the Heritage facility vacuum truck cleaning was found to be more effective than sweeping. The average increase to infiltration rate was 89% considering all sites and 30% when sites B and D were removed from analysis. Both of these numbers are greater than the increase due to sweeper cleaning. The addition of pressurized air resulted in additional material being removed from the pores, increasing infiltration recovery rates. As compared to the Heritage facility, at the College Street facility vacuum truck cleaning was less effective, only restoring infiltration rates at three of the six sites with an average increase to infiltration rates of 17%. During post cleaning observations of the site, material was observed to still be trapped in the PCP, indicating that there was a limit to the extent that vacuum truck cleaning can be used on clogged PCP.

Based on the results from the Heritage facility, it can be concluded that cleaning using a vacuum truck results in a better restoration of infiltration rates than a sweeper. It appears though that regular (bi-annual) sweeping at the Heritage facility reduced areas of clogging, allowing for increased restoration. Likewise, severe clogging at the College Street facility was likely due to the lack of regular cleaning. This clogging reduced the effectiveness of vacuum truck cleaning. These observations indicate that it would be beneficial to clean PCP facilities regularly; twice a year (in spring and in fall) preferably using a vacuum truck. If that is not feasible, cleaning with a sweeper is expected to be better than no cleaning at all.

4.5.2.2 <u>Spot Cleaning</u>

Results of spot cleaning methods performed at the College Street and Heritage facilities are presented in Figure 4.7 and Figure 4.8, respectively. Sites are reported with their numerical identifier, pre-cleaning infiltration rate, post-cleaning infiltration rate and percentage restoration, all infiltration rates are in in/hr. Figures have been modified for the Heritage facility to better show the area where cleaning was performed.

Pre-cleaning infiltration rates at a given site are generally similar for all cleaning methods. The largest variation was observed at College Street at site 2 which ranged from 700 to 1,120 in/hr (0.53 to 0.83 cm/sec). At the Heritage facility the largest variation in pre-cleaning infiltration rates was found at site 3, ranging from 42 to 1,000 in/hr (0.03 to 0.72 cm/sec). Sites at both facilities were found to vary greatly with pre-cleaning infiltration rates ranging from 82 to 1,160 in/hr (0.006 to 0.83 cm/sec). These values indicate that cleaning procedures were tested on a wide variety of clogging conditions.

Pressure washing increased infiltration rates at all sites except for site 1 at the Heritage facility, with increases of 4% to 591% and an average increase of 85%. The largest increases to infiltration rates were observed in areas such as College Street 4 and Heritage 4-6 indicating that pressure washing can restore areas of severe clogging. At site 8 pressure washing was observed to cause a small amount of raveling; however at all other sites no such raveling was observed. Vacuuming was also found to restore infiltration rates from 4% to 28% with an average value of 10%. However, College Street sites 2 and 4 and Heritage sites 4 and 6 showed no increase, indicating that vacuuming alone cannot remediate severe clogging. Combined pressure washing and vacuuming resulted in restoration of infiltration rates at all sites including severely clogged sites with increases to infiltration rate ranging from 6% to 1,070% and averaging of 100%. Results of spot cleaning indicate that different methods result in different treatment efficiencies. Although these methods are impractical for facility wide cleaning operations pressure washing and combined pressure washing and vacuuming were found to successfully remediate severely clogged areas that street sweepers and vacuum trucks could not clean. Pressure washing followed by vacuuming would be ideal to prevent the migration of material into PCP; however, pressure washing alone can be effective if small areas (compared to the overall site) are to be cleaned.

4.5.3 Comparison to Previous Studies

4.5.3.1 Long-term Monitoring

Reduction in infiltration rate of 87% in the field observed Henderson et al. (2009) compare reasonably well with values observed in this study with the College Street and Heritage facilities showing 59% and 26% reductions, respectively. Laboratory reductions were observed to be approximately 40% and 30% in specimens exhibiting clogging (Young and Grasley, 2008; Deo et al., 2010). Both of these results fall within the range of field observations indicating that laboratory studies can be used to estimate field performance of PCP.

4.5.3.2 Infiltration Recovery Methods

Field testing of vacuum cleaning performed by Henderson et al. (2009) showed post-cleaning infiltration rates were 100 times greater than pre-cleaning infiltration rates. In another study, cleaning increased infiltration rates 10, 56 and 66 fold for vacuuming, pressure washing and combined methods respectively (Chopra et al., 2010). These increases to infiltration rate were considerably larger compared to the results of this study; with vacuuming restoring infiltration rates by 1.1 times original value on average, pressure washing restoring infiltration rate by 1.85 times original value and combined methods 2 times original value. This difference between the efficiency of cleaning methods is significant. In results reported by Henderson et al. (2009) cleaning was performed one year after construction, whereas in this study cleaning was not investigated until after the second year of operation. The additional time between construction and cleaning in this study likely allowed material to be forced into the PCP, resulting in reduced cleaning efficiency. Different hydraulic testing conditions (saturated hydraulic conductivity vs. unconstrained infiltration) make direct comparisons with the second study difficult and could explain the observed difference; however, the effectiveness of each cleaning method compared to one another (vacuum vs. pressure washing) is similar for both studies.

4.6 CONCLUSIONS AND DISCUSSION

The objectives of this study were to observe the performance of PCP in the field; determine the effectiveness of cleaning methods to restore infiltration rates, and compare field observations to available laboratory results when possible. This was accomplished by monitoring two PCP sites in Vermont over about a year long period and testing various cleaning methods at these sites. At each site, several locations were monitored and tested.

Infiltration rates measured at the PCP facilities decreased over the one year period with average reductions of 59% for the College Street facility and 26% for the Heritage facility. Reductions to infiltration rate over time were gradual, indicating that the clogging process occurred consistently over both sites irrespective of the season. Differences were observed between sites within and outside of the wheel track of vehicles, clogging was also observed to be more severe in locations where trucks regularly traveled. Previous studies that simulated clogging in laboratory found infiltration rates decreasing in a more or less linear fashion as the clogging material was added, with final reductions ranging from 20% to 40%. This suggests that the real world clogging of PCP can be simulated in laboratory studies.

Facility wide cleaning operations included street sweeping and vacuum truck cleaning, which resulted in increases to infiltration rate of 21% and 30%, respectively, and both methods are recommended for regular cleaning operations. Spot cleaning methods were also investigated with increases of 85% after pressure washing, 10% after vacuuming and 100% after pressure washing followed by vacuuming. Cleaning by pressure washing was found to be effective at restoring infiltration rates in areas of severe clogging, however the method forces material into the gravel subbase potentially clogging it in the future. Vacuuming along with pressure washing is preferred as it is able to restore infiltration rate and remove material from PCP. These field results of infiltration rates were found to yield substantially less recovery of infiltration rates compared to previous laboratory studies. Nonetheless, less than 15% of the total PCP area was clogged in both facilities, and in general the infiltration rate of the PCPs remained greater than the infiltration needed to accommodate a 100-yr storm event, except in the severely clogged regions.

4.7 REFERENCES

ACI Committee 522, "Report on Pervious Concrete 522R-10," American Concrete Institute, Farmington Hills, Mich., 2010.

Bean, E. Z., Hunt, W. F., and Bidelspach, D. A. (2007). "Field Survey of Permeable Pavement Surface Infiltration Rates." J. of Irrigation and Drainage Engineering: 133(3), 249-255.

Chopra, M., Kakuturu, S., Ballock, C. Spence, J. (2010). "Effect of Rejuvenation Methods on the Infiltration Rates of Pervious Concrete Pavements" Journal of Hydraulic Engineering, Vol. 15, No. 6, June 2010.

Deo, O., Sumanasooriya, M., and Neithalath, N. (2010)."Permeability Reduction in Pervious Concretes due to Clogging: Experiments and Modeling" Journal of Materials in Civil Engineering, Vol. 22, No. 7, July 1, 2010.

Guthrie, W. S., DeMille, C. B., Eggett, D. L. (2010). "Effects of Soil Clogging and Water Saturation on Freeze–Thaw Durability of Pervious Concrete" Transportation Research Record: Journal of the Transportation Research Board No. 2164, Washington, DC, pp. 88-97.

Henderson, V., Tighe, S.L., and Norris, J. (2009). "Pervious Concrete Pavement: Integrated Laboratory and Field Study." Transportation Research Record: Journal of the Transportation Research Board. No. 2113, Washington, DC, pp. 13-21.

Joung, Y., and Grasley, Z. C. (2008). "Evaluation and Optimization of Durable Pervious Concrete for Use in Urban Areas." *Technical Report for the Texas Transportation Institute, Texas A&M University.* February. College Station, TX

Montes, F. and Haselbach, L.M. (2006). "Measuring Hydraulic Conductivity in Pervious Concrete." *Environmental Engineering Science*: 23(6), 960-969.

Olek, J., Weiss W.J., Neithalath N., Marolf A., Sell E., and Thornton, W.D. (2003). "Development of quiet and durable porous Portland cement concrete paving materials. "Final Report SQDH 2003-5. West Lafayette, IN: Purdue University.

Portland Cement Association (PCA) (2003). "Building Green with Concrete: Points for Concrete in LEED 2.1, IS312." Portland Cement Association, Skokie, IL.

Suozzo, M., and Dewoolkar, M.M. (2011). "Evaluation of Strength and Hydraulic Testing Methods of Pervious Concrete" Submitted to ACI Materials Journal November 2011 (In Review).

Tennis, P. D., Leming, M. L., and Kiefer, C. J. (2004). "Pervious Concrete Pavements." *Portland Cement Association*, Skokie, IL.

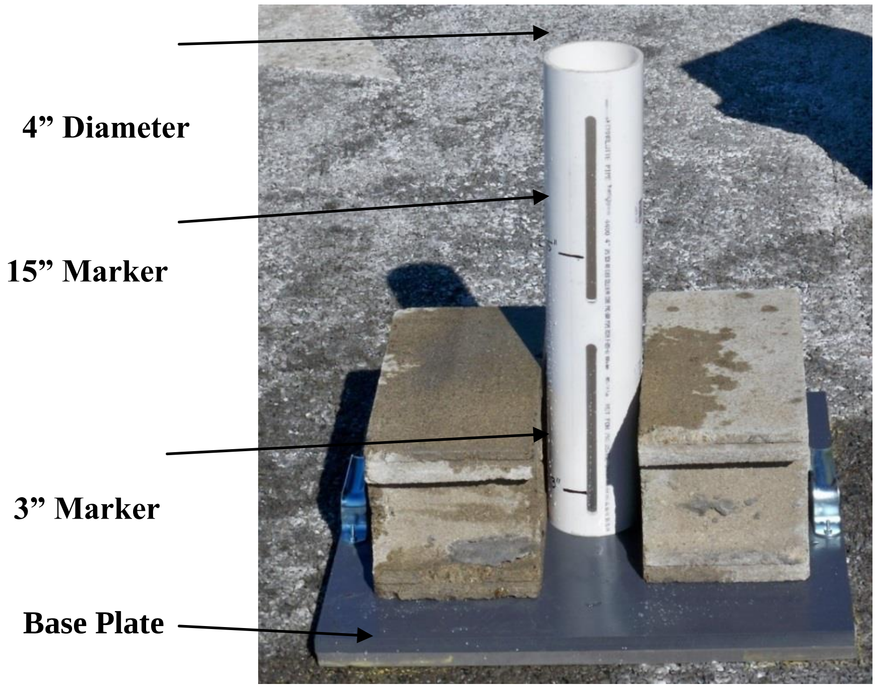
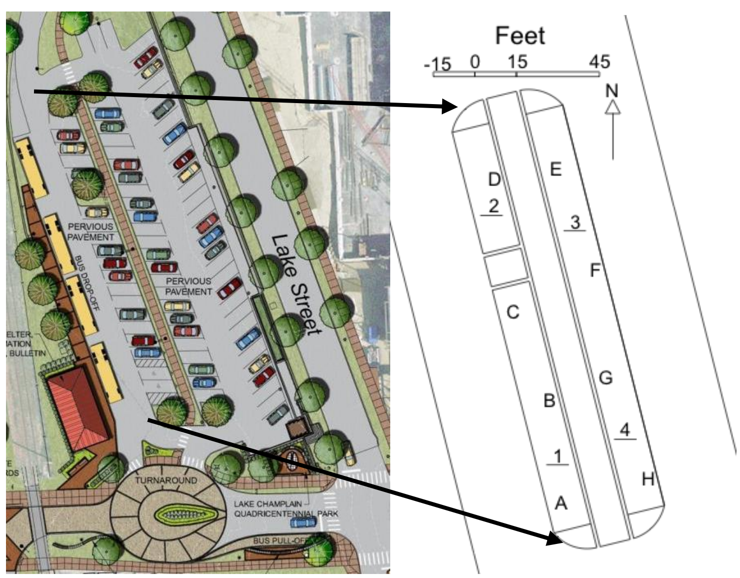


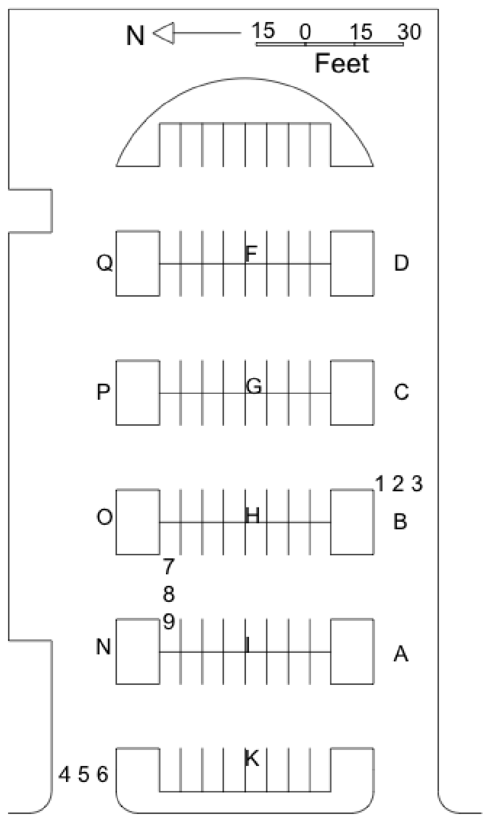
Figure 0.1 - Falling head infiltrometer in use.



(a) **(b)**

Figure 0.2 - College Street PCP facility, (a) Overview of facility, and (b) Location of testing sites.

(a)



(b)

Figure 0.3 - Heritage PCP parking facility, (a) Photograph of the facility, and (b) Location of testing sites.

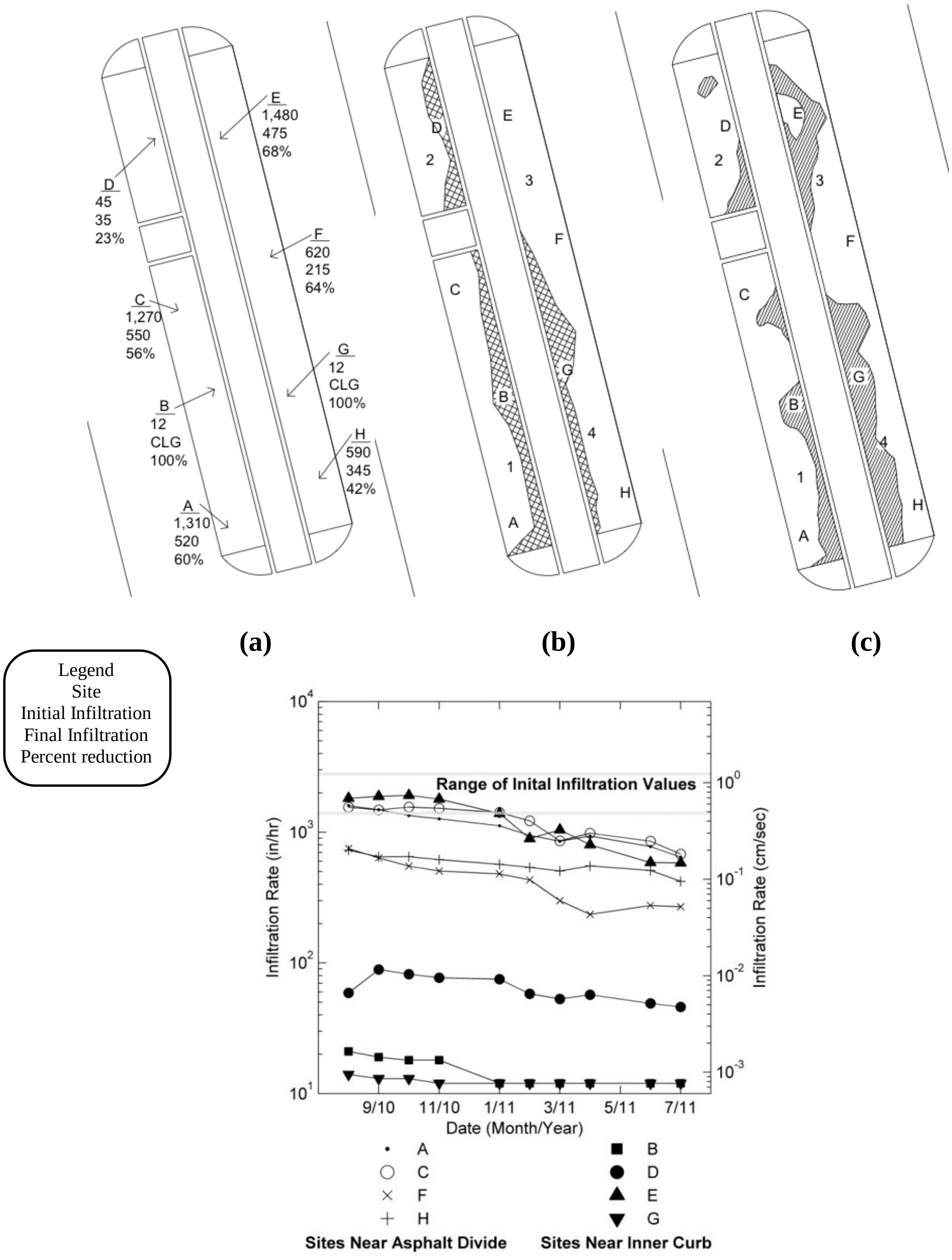


Figure 0.4 - Results of field monitoring at College Street facility. (a) Infiltration sites, (b) Visual signs of clogging, (c) Visual signs of raveling, and (d) Infiltration measurements.

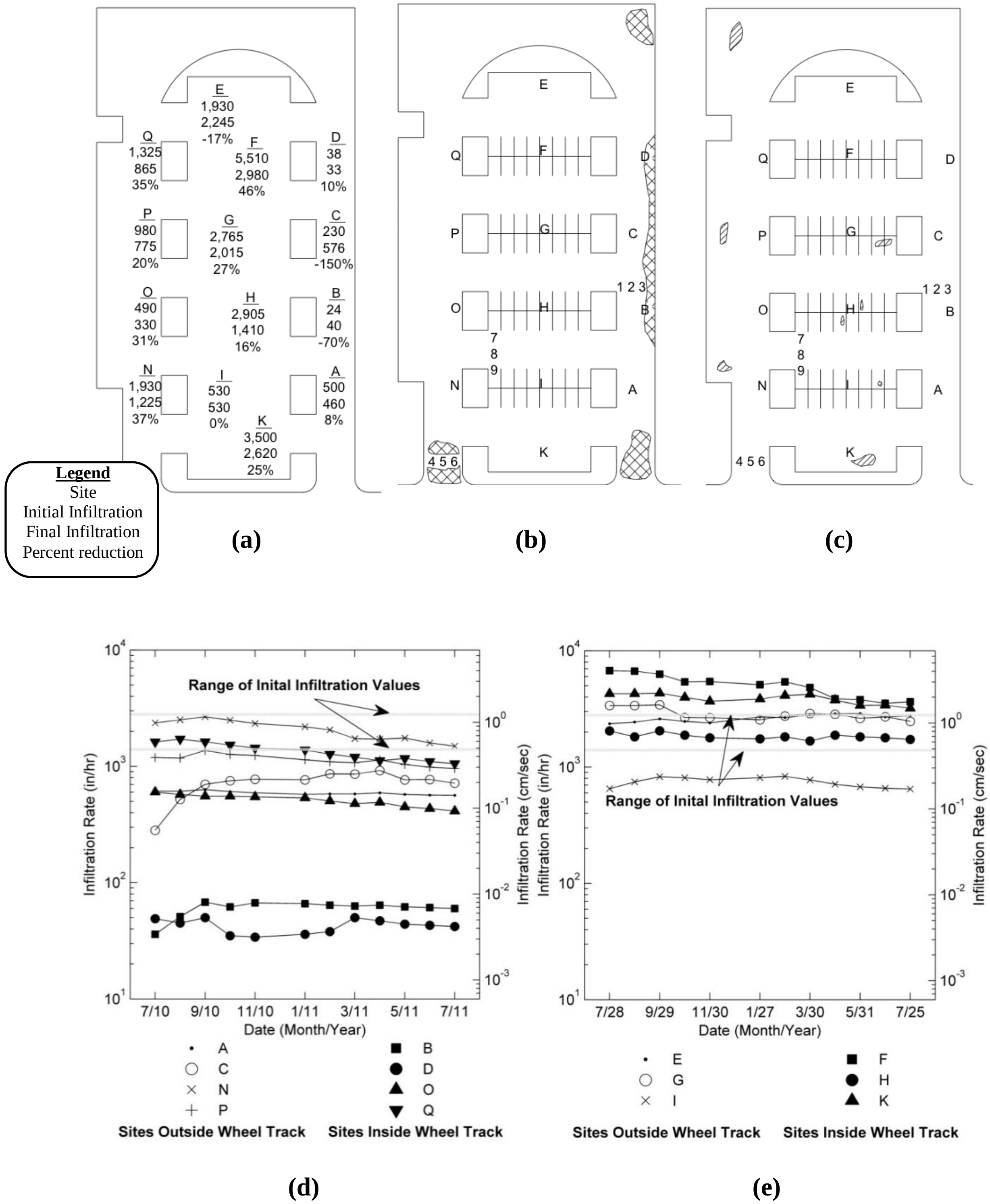
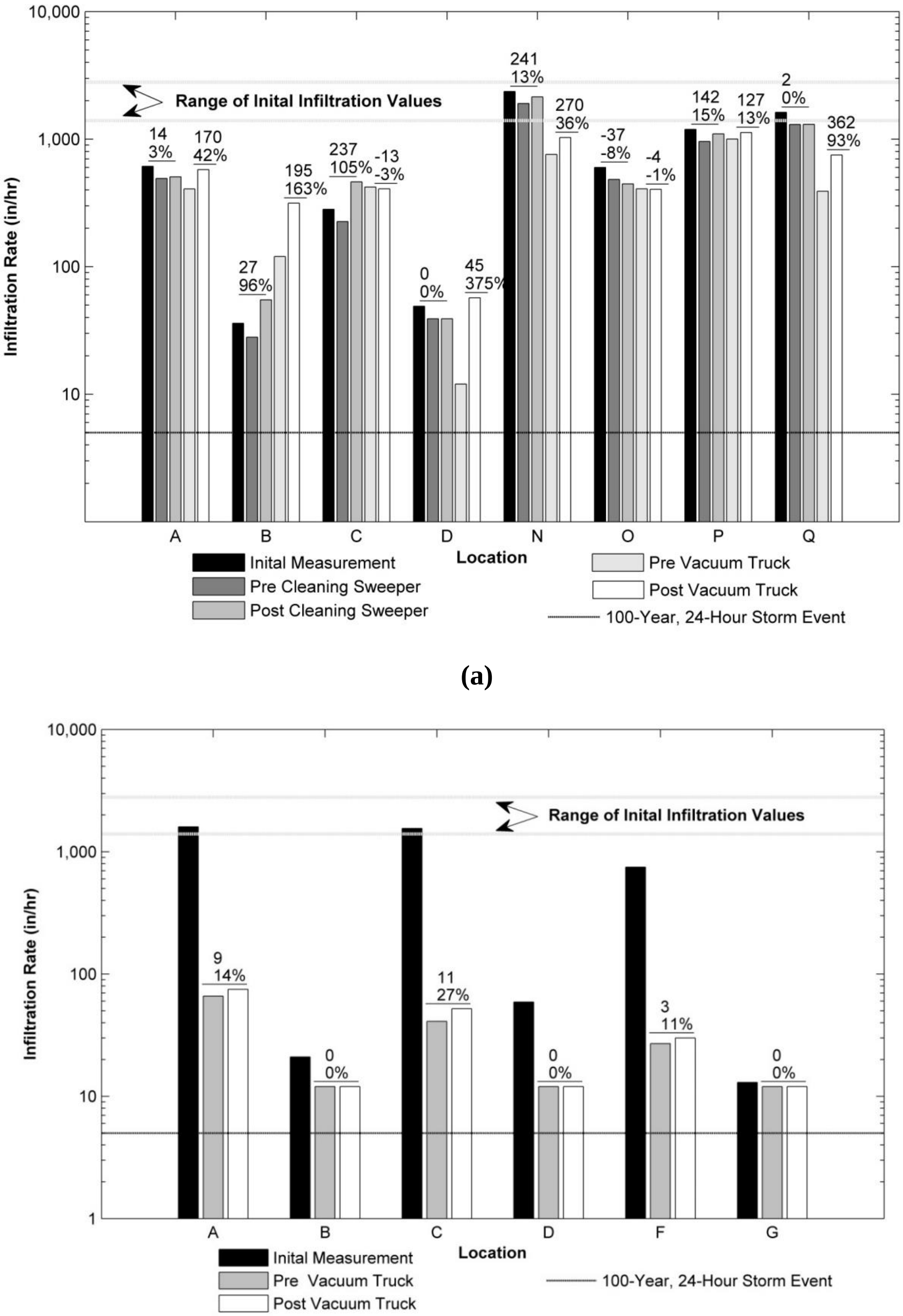


Figure 0.5 - Results of field monitoring at Heritage facility. (a) Infiltration sites, (b) Visual signs of clogging, (c) Visual signs of raveling, (d) Infiltration measurements in travel lanes, and (e) Infiltration measurements in parking areas.



(a)

(b)

Figure 0.6 - Results of Maintenance operations (a) Heritage, (b) College Street lot.

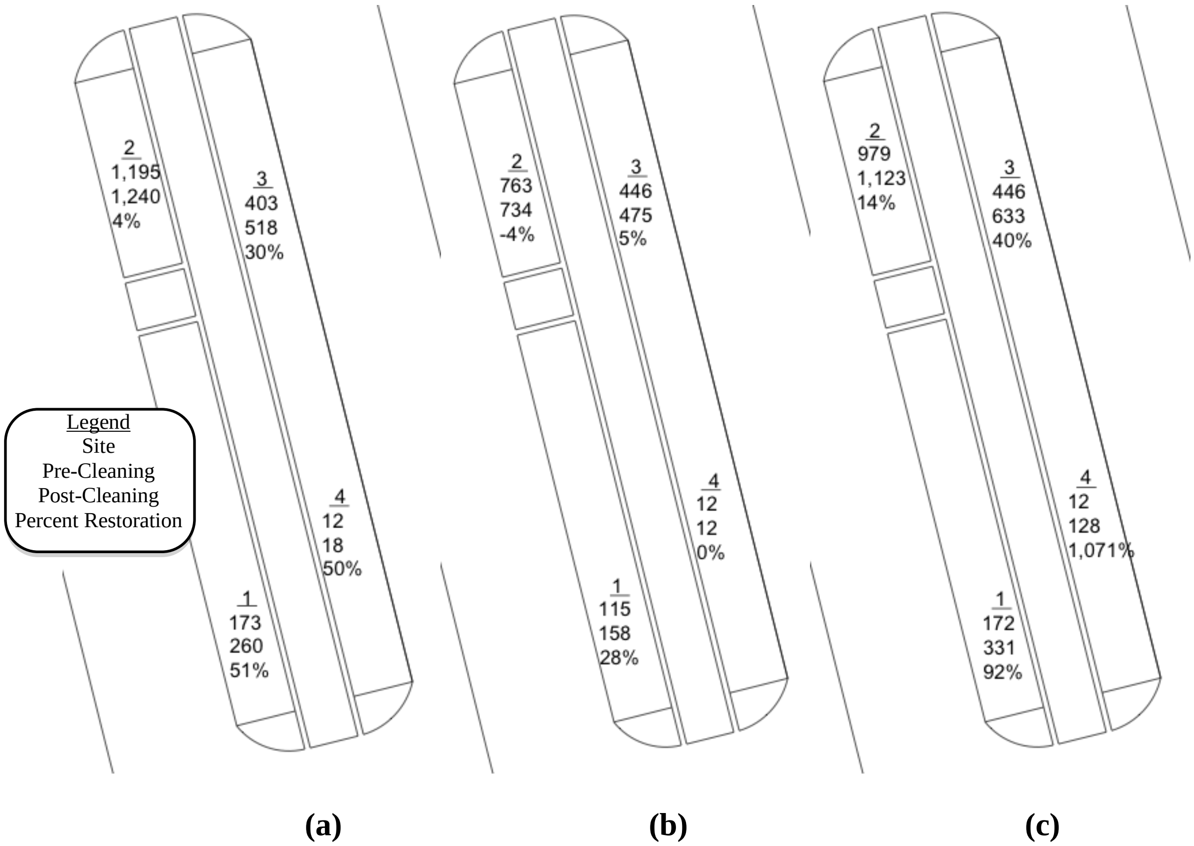
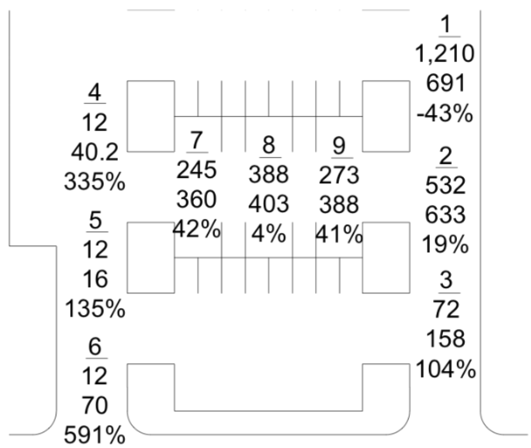
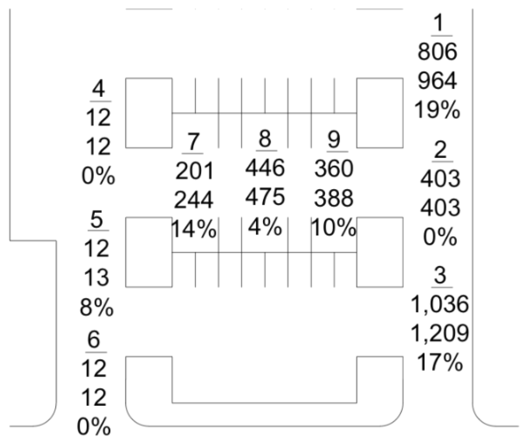


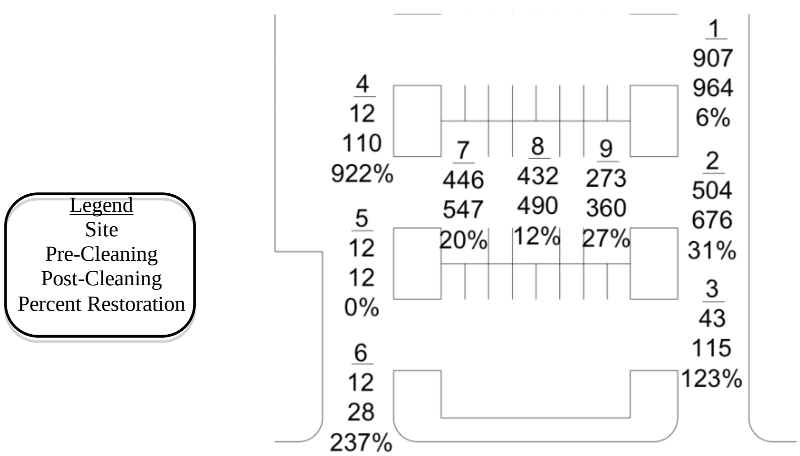
Figure 0.7 - Effects of various cleaning methods on infiltration rate at the College Street facility (a) Pressure washing, (b) Vacuuming and (c) Combined



(a)



(b)



(c)

Figure 0.8 - Effects of various cleaning methods on infiltration rate at the Heritage facility. (a) Pressure washing, (b) Vacuuming and (c) Combined.

CHAPTER 5

DURABILITY OF PERVIOUS CONCRETE WITH FLY ASH SUBJECTED TO FREEZE-THAW AND SALT EXPOSURE, IN A SIMULATED FIELD ENVIRONMENT

5.1 ABSTRACT

This study investigates the durability of pervious concrete with fly ash to freeze-thaw and salt exposure in a field representative environment. Laboratory pervious concrete cylinders were prepared with 0, 10, 20 and 30% fly ash replacement. Along with freeze-thaw testing, specimens were tested for void ratio, compressive strength, and hydraulic conductivity. Freeze-thaw testing was conducted using a slow freeze-thaw cycle, of one cycle per day, in drained condition. Salt solution concentrations used included 0, 2, 4, 8, and 12% sodium chloride. The measured void ratio, compressive strength, and hydraulic conductivity of the mixes were all within the usable range for pervious concrete. The results from freeze-thaw testing suggest that for all concentrations of salt solution, 10 and 20 % fly ash replacement improved freeze-thaw durability. Specimens with 30% fly ash showed more damage than that of the 0% control. The greatest damage from salt solutions was seen in 8, 4, and 2% concentrations respectively. Water and 12% salt solution showed little damage, across all mix designs.

5.2 INTRODUCTION

Pervious concrete, with its ability to act as both a structural pavement, and stormwater mitigation system, has been identified as a best management practice (BMP) to efficiently manage stormwater runoff and reduce stormwater pollution (ACI, 2010; EPA, 2000). Pervious concrete pavements are ideal for sites with limited space, where traditional stormwater collection systems may not be viable. They can particularly be useful as parking lots and sidewalks. The purpose of pervious concrete as a stormwater management system is to allow water to flow through, and collect in its underlying holding layer, where it infiltrates into the subsoil, or discharges offsite. Pervious concrete has several advantages over conventional pavements. The infiltration of the water through its interconnected pores can reduce hydroplaning potential, improve skid resistance, and reduce the runoff potential (Tennis et. al, 2004). Pervious concrete is able to capture the "first flush" (the first inch of rainfall), which contains the most polluted stormwater, reducing sediment loading, and limiting flash flood potential.[3] Pervious concrete is able to eliminate potential pollutants that otherwise would make their way to nearby streams or wetlands (Schuler, 1987). Pervious concrete has been shown to remove up to 95% of total suspended solids (TSS), 65% of total phosphorous (TP), 85% of total nitrogen (TN), and 99% of metals from stormwater runoff (Shuler, 1987). On the other hand, while open void spaces in pervious concrete allow collection of stormwater, they also make it vulnerable to clogging, and reduces its strength and durability to environmental factors such as freezing and thawing.

Pervious concrete is typically described as a zero-slump, open graded material consisting of portland cement, coarse aggregate, little or no fine aggregate, admixtures, and water (ACI, 2010). The absence or small amount of fine aggregate leads to open voids between cement covered aggregate. Uniformly graded aggregate is typically used to maximize the void space, to create hydraulically connected paths for water to flow. Typical void ratio for pervious concrete is 18-35% (ACI, 2010; Tennis et. al, 2004). This range is considered ideal to provide enough strength, while allowing for sufficient hydraulic conductivity. Compressive strength of pervious concrete ranges from 2.8 to 28 MPa (400 to 4000 psi) (ACI, 2010). Hydraulic conductivity typically ranges from 0.2 to 1.2 cm/sec (280 to 1,680 in/hr) (NRMCA, 2004). Such high infiltration makes pervious concrete for excellent stormwater collection. The relationship between hydraulic conductivity, compressive strength, void ratio, and density has been shown to be directly related (McCain and Dewoolkar, 2010; Wanielista and Chopra, 2007; Meininger, 1988). With higher void ratio, hydraulic conductivity increases, and compressive strength decreases. Density, which is closely related to compaction energy, increases with compressive strength, and decreases with hydraulic conductivity and void ratio (Wang et al., 2006).

The large void spaces and thin cement paste leave pervious concrete susceptible to freeze-thaw type damage, an issue that has limited its use in cold regions. The presence of water, by design, puts pervious concrete in a vulnerable state. When fully saturated in water and frozen, the water expands forcing the aggregates apart. The standard test for freeze-thaw durability, ASTM C666 (2008) *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*, consists of cycling fully saturated concrete specimens 7 times a day, until 300 cycles. Mass lost of the samples is then measured, with 15% loss considered as failure (Schaefer et al., 2006). Tests have shown that with the addition of sand, pervious concrete can withstand over 300 freeze-thaw cycles, passing the ASTM C666 standardized test for durability (Kevern et al., 2008b). Other investigations have studied adding admixtures and fibers or changing the water to cement ratio (w/c), coarse aggregate, and moisture conditions (Wang et al., 2006; Kevern et al., 2008b; Ghafoori and Dutta, 1995; Kevern et al., 2008a; Yang et al., 2006; Yang, 2011).

The American Concrete Institute committee 522 report (2010) does not recommend the ASTM C666 test for pervious concrete, because the test does not represent field conditions well. The fully saturated test condition, and the rapid cycling of freeze-thaw make for an unrepresentative testing environment. As an alternative, testing under drained condition and one freeze-thaw cycle per day has been recommended for pervious concrete by some researchers (Yang, 2011; Olek and Weiss, 2003). It has been suggested that increased saturation is needed for damage and that below the critical saturation levels, no damage would occur in pervious concrete from freezing and thawing (Yang et al., 2006). Critical saturation in conventional concrete is expected to be about 60% for the freeze-thaw damage to occur (Litvan, 1973; Vuorinnen, 1970). Additionally, frozen water in the large pores of pervious concrete acts to create negative vapor pressures, drawing the liquid water through the cement paste, causing scaling damage (Harnik et al., 1980).

In cold climates, road salts are used to melt snow and ice on pavements. The commonly used salts are sodium chloride and calcium chloride. Salt exposure in concrete can lead to formation of salt crystals in the pores, and at high concentrations can change the chemical composition in the cement paste (Darwin et al., 2007). The crystals can put additional pressure on the cement paste, dislodging it from the aggregates. The chemical reaction causes the cement paste to lose its structure, and the bonds can be destroyed (Cody et al., 1996; Lee et al., 2000).

For sodium chloride, the most commonly used road salt, application is usually targeted at 23%, at which the freezing point is the lowest (Cloutier and Newbury, 2009). Studies have shown that a 2-4% percent solution of salt causes maximum scaling (cement paste to be dislodged) in saturated conditions, and that above and below this range less scaling is expected (Verbeck and Klieger, 1957; Marchand et al., 1999). Conversely, for the wetting-drying condition, the amount of damage increases as the concentration of salt increases (Cody et al., 1996).

Freeze-thaw testing conducted with a 3% sodium chloride solution showed that as the water in solution freezes, the concentration of the unfrozen solution can rise to nearly four times the original concentration (Chan et al., 2007). This effect, known as freeze concentration, is believed to aid in the process of supercooling, which occurs when the freezing point of the solution is depressed because of the increased salt concentration, until the point where the phase shift in the water does occur, and the solution freezes at much greater pore pressures (Harnik et al., 1980). It has been argued that the application of deicing salts allows the degree of saturation in conventional concrete to exceed the amount normally attainable with pure water (Harnik et al., 1980). Additionally salt crystallization is identified as a source of pressure in large pores in concrete, by both physical forces and hydraulic pressures, as it draws water out of smaller pores. It has been shown that in the ASTM C666 rapid freeze-thaw testing the use of air entraining admixtures can significantly improve the deicing scaling resistance (Pigeon and Pleau, 1995). However, it has also been shown that in the slower, one freeze-thaw cycle per day testing, durability does not increase with air entraining admixtures (Yang, 2011).

The study presented here investigated the use of fly ash as a cementitious additive in pervious concrete, in an attempt to improve freeze-thaw durability and salt resistance of pervious concrete. Fly ash is a byproduct of the combustion of coal for generation of electricity. Fly ash is a small spherical particle, typically 0.2-10 μm, which occurs when mineral impurities fuse during combustion (Chindaprasirt et al., 2005). This material is categorized based on ASTM 618

(2008), *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*, with most concretes incorporating class C or F fly ash. Fly ash is the most commonly added supplementary cementitious material, with about 50% of all ready-mix concrete incorporating some amount of fly ash (PCA, 2002). Fly ash addition in conventional concrete has been shown to reduce water demand, similar to chemical water reducing admixtures (Helmuth, 1987). The small spherical particles act to lubricate the cement, improving workability, and can extend the set time (Chindaprasirt et al., 2004). Fly ash has been shown to increase the long term compressive strength of conventional concrete, but requires curing beyond the typical 28 days (PCA, 2002; Chandaprasirt et al., 2004). The smaller particle size relative to cement allows for a greater distribution of particle sizes, which can act to reduce the pore sizes of the cement paste (Chindaprasirt et al., 2005). Fly ash with increasingly finer particles were shown to further reduce porosity, pore size, and improve strength and workability (Chindaprasirt et al., 2005; Chindaprasirt et al., 2004). While fly ash has been used in conventional concrete for some time, little is known about its possible effects when incorporated into pervious concrete.

5.3 RESEARCH SIGNIFICANCE

Significant research and development have occurred for pervious concrete, but its acceptance in cold climates is still limited. Vulnerability to freeze-thaw and salt exposure has led to uncertainty about its long term performance. Additionally, the current standardized freeze-thaw testing procedure is not recommended for pervious concrete, as it is not representative of field conditions. This study employed testing processes that are more representative of field conditions to determine the effects of fly ash on freeze-thaw durability and deicing salts exposure of pervious concrete. The use of pervious concrete itself is considered a best management practice in stormwater management; possibility of substituting cement with a waste product such as fly ash promotes sustainability even further.

5.4 EXPERIMENTAL INVESTIGATION

The specific objective of this study was to investigate the resistance of pervious concrete to freeze-thaw and salt exposure. The pervious concrete mix design included varying amounts of fly ash. Pervious concrete specimens were prepared and tested to mimic field conditions, including air curing, once daily freeze-thaw cycles, and allowing for the specimens to be fully drained. By varying the amount of fly ash, and the concentration of salt in solution, the damage and durability of the pervious concrete was studied.

5.4.1 Mix Designs and Sample Preparation

The mix designs used in this study were based on the mix used at the Park and Ride site in Randolph, Vermont, which was constructed in Fall 2008. All mix designs used the same 10 mm (3/8") crushed ledge as its course aggregate, with no fine aggregate. Type I-II portland cement was used for all mixtures. Chemical admixtures used included an air entraining agent (AEA), high range water reducer (HRWR), viscosity modifying admixture (VMA), and hydration controlling admixture (HCA). Class F fly ash (FA) was used as an additional alternative cementitious material. Different levels of cement replacement with fly ash were used, to investigate fly ash's effects on the pervious concrete. Details of mixes studied are summarized in Table 5.1.

Pervious concrete specimens were prepared in accordance with ASTM C192 (2008), *Practice for Making and Curing Concrete Test Specimens in the Laboratory*. The specimens were prepared by compacting with a 10 mm (3/8") tamping rod, in two lifts with 25 strokes per layer. Samples were cast into cylinders with diameter 10.2 cm (4") and length 20.3 cm (8"). Specimens were moist cured at 23 ± 2°C for 28 days, but were demolded after the initial 7 days, to better replicate field conditions (Tennis et al., 2004; Yang, 2011). The specimens were used for testing fully drained in slow freeze-thaw chambers, to better replicate field conditions, rather than the typical ASTM C666 *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing* (Yang, 2011). Conventional engineering properties of compressive strength, hydraulic conductivity, and void ratio were also measured.

5.4.2 Void Ratio

The void ratio of the specimens was found by taking the difference in weight of an oven dried sample, and one submerged in water, using Equation 1 (Park and Tia, 2004). Void ratio testing took place on three specimens from each mix design, with each specimen tested three times.

$$V_r = \left[1 - \left(\frac{M_w - M_d}{\rho_w * Vol}\right)\right] \tag{1}$$

where,
V_r = void ratio,
M_w = mass in water (g),
M_d = dry specimen mass (g),
ρ_w= density of water (g/cm^3),
Vol = volume of specimen (cm^3).

5.4.3 Compressive Strength

Compressive strength was determined in general accordance with ASTM C39 (2008), *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. Specimens used elastomeric pad caps in accordance with ASTM C1231 (2008), *Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders*. Compressive strength testing was done after the full 28 day curing time, and was conducted on five replicate specimens.

5.4.4 Hydraulic Conductivity

Hydraulic conductivity tests on the pervious concrete were performed in general accordance with the procedures outlined by Olek and Weiss (2003). Samples were wrapped in synthetic rubber sheeting, and a rigid cylindrical mold to prevent lateral flow. The samples were back saturated to remove air from the voids. A downstream head was maintained above the sample height to ensure full saturation. The flow of water was measured as it fell between two known head locations, 38.1 cm (15 in) and 7.6 cm (3 in). Previous research has shown that for these head values, laminar flow is maintained, allowing the application of Darcy's Law to interpret the test results (Montes and Haselbach, 2006). The hydraulic conductivity can be found using the following equation:

$$k = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right) \tag{2}$$

where,

k = hydraulic conductivity (cm/s),
a = cross-sectional area of the standpipe (cm^2),
L = length of specimen (cm),
A = cross-sectional area of specimen (cm^2),
t = time for water to drop from h_1 to h_2 (s),
h_1 = initial water level (cm),
h_2 = finial water level (cm).

5.4.5 Freeze-Thaw Testing

Freeze-thaw testing was conducted, with the intent to replicate field conditions. A single day freeze-thaw cycle was used for this testing, after 28 days of air curing. The cycle includes 16 hours at -20°C (-4°F), and 8 hours at 25°C (77°F). The testing also included a wetting-drying phase, where the specimens were submerged in solution for the last hour of the thawing segment, then removed and allowed to drain for the freezing cycle. The drained specimens are representative of a functioning pervious concrete system. Solutions used include water and sodium chloride solution at 2, 4, 8, and 12% by weight. Solutions were prepared every 10 days to ensure desired concentrations. Testing was conducted with five replicate specimens for each mix design and salt combination. Specimens were measured for mass lost every 10 days, with testing continuing until 100 days. Mass lost of 15% was considered failure for this test as recommended (Schaefer et al., 2006).

5.4.6 Scanning Electron Microscope

The particle size of type I-II cement on average is between 10-20 μm (Bentz et al., 2008). Class F fly ash has an average particle size between 0.2-10 μm (Chindaprasirt et al., 2004). In an attempt to observe a difference in the cement pastes containing varying amounts of fly ash, a JEOL 6060 scanning electron microscope (SEM) was used. Through the use of the SEM, the cement covering individual aggregates could be observed with high resolution.

5.5 EXPERIMENTAL RESULTS AND DISCUSSION

The void ratio, hydraulic conductivity, compressive strength, and freeze-thaw durability measurements are presented and discussed in this section. Whenever possible, comparisons to similar measurements reported in the literature are also made.

5.5.1 Engineering Properties

The typical properties that characterize pervious concrete performance are void ratio, hydraulic conductivity, and compressive strength. The combination of these measures provides a qualitative indication of the performance of the mix design. It is common to see relationships between these properties, where a mix with high void ratio will generally have high hydraulic conductivity, and low compressive strength. Table 5.2 summarizes results from these tests. Compressive strength values are the average of five samples per mix design, and hydraulic conductivity and void ratio are the average of three repeat tests on three samples per mix design.

Void ratio, which is typically between 18-35%, was consistently high across the mix designs, averaging 0.351(ACI, 2010; Tennis et al., 2004). Void ratio can be affected by the aggregate to cement (a/c) ratio and compaction energy. With high a/c ratio, there is a thinner cover of cement paste surrounding each aggregate, allowing for a greater void space between aggregates. Studies of a/c ratios as high as 5, in conjunction with water to cement (w/c) ratio of 0.35 - 0.39, had provided the greatest strength while not sacrificing infiltration (Wanielista et al., 2007). For the study reported here, the a/c ratio was 4.4, which corresponded well with the high void ratio.

The hydraulic conductivity measurements showed the mixes to be above the typical range of 0.2 - 1.2 cm/s (280 – 1,700 in/hr) (NRMCA, 2004). The average hydraulic conductivity for all specimens was 1.24 cm/s (1,758 in/hr), with the lowest measurement of 1.14 cm/s (1,616 in/hr). The relationship between hydraulic conductivity and density is shown in Figure 5.1. Typically, lower density samples show an increase in hydraulic conductivity (McCain and Dewoolkar, 2010). In this study however, that relationship is not seen, possibly due to the narrow range of density. Figure 5.2 shows the results of hydraulic conductivity of this study with some of those found in literature. Mixes containing fly ash showed somewhat lower hydraulic conductivity when compared to the mix with only portland cement. This deviation may be due to an increase in workability of the cement paste with the addition of fly ash, allowing for greater settling and impacting of the voids by cement paste than seen in Mix 1 with no fly ash (PCA, 2002). Nonetheless, all samples show high infiltration potential, above typical ranges, and show the mixes to be in the practical range for field applications.

The measured compressive strength was well within the recommended ranges of 3.5-28 MPa (500 – 4,000 psi), but falls below the recommended value of 17 MPa (2,500 psi) (Tennis et al., 2004; NRMCA, 2004). The average compressive strength of all samples tested was about 10.1 MPa (1,465 psi), with the minimum measurement of 7.97 MPa (1,156 psi). The relationship between compressive strength and density can be seen in Figure 5.3. The compressive strengths did increase slightly with density increase as is typical of pervious concrete (McCain and Dewoolkar, 2010). Figure 5.4 shows the compressive strength values found in literature with those found in this study. Failure during testing did not appear to be solely through the cement paste, indicating it was not exclusively a weakness in the paste that resulted in lower than expected values. Mix 4, with 30% fly ash replacement showed lower compressive strength, and may indicate the curing of the fly ash was incomplete. Fly ash is expected to require curing time in excess of 28 days, to develop its full strength (Chindaprasirt et al., 2004).

Overall, all mix designs fell within acceptable ranges for void ratio, hydraulic conductivity, and compressive strength. The results indicate that the mixes are on the high side for void ratio and hydraulic conductivity, and subsequently the lower end of compressive strength. A thicker cement paste may benefit with an increase in strength, while maintaining acceptable void ratio and hydraulic conductivity. Possible solutions could be the addition of fibers, or fine aggregate to the mix, or increasing the cement content relative to the aggregate.

5.5.2 Freeze-Thaw Durability

Freeze-thaw tests were conducted to compare durability of the studied mix designs containing varying amounts of fly ash replacement. Five concentrations of sodium chloride salt solution (0, 2, 4, 8, and 12% by weight) were used for wetting-drying during testing. All testing was done using a slow, one cycle per day process, in an attempt to more closely approximate field conditions. The specimens were kept submerged in solution for the last hour of the thaw, and then be allowed to drain while freezing. Specimens were allowed to drain because that would be expected in a properly functioning pervious concrete pavement. Failure was considered when a specimen lost 15% mass. The specimens were subjected to a maximum of 100 freeze-thaw cycles. For all samples tested, damage was seen to originate from the bottom center of the sample. The location of the most damage corresponded to the place with the greatest level of saturation, as the samples would dry from top and outside first. This pattern indicates the damage is directly related to the level of saturation present. The observed damage appeared to be scaling of the cement paste between aggregates, with little to no damage to the aggregate. Figure 5.5 summarizes the results of the salt solution exposure as the average of all specimens from of all mix designs. Each data point represents 16 specimens. Of the solutions tested, 8 and 4% salt were the most damaging, with the samples in 8% solution failing before the end of testing. This can be seen in Figure 5.6, which shows the effects of salt concentrations on individual mix designs. The least damage was seen in the 12% salt and no salt samples, with 2% falling between the two groups. The 12% solution, which was expected to be highly damaging, showed the least mass lost. One possible reason is as the temperature lowers, water is frozen out of solution, leaving a higher concentration salt solution, and further depressing its freezing point. The longer freeze time may have allowed more drying, lowering the saturation as compared to the other specimens. It appears that the higher the salt concentration, the greater the damage, up until the freezing time is affected. Had the samples of the highest concentration seen the same amount of time in the frozen state, the damage likely would have exceeded that of lower concentrations.

Figure 5.7 compares the results of freeze-thaw testing for different fly ash percentages, for all the salt solutions. Each data point represents 20 specimens. Figure 5.7 shows that for all the solutions used, mixes with 10% and 20% fly ash replacement outperformed the control specimens with no fly ash. Mixes of 0 and 30% fly ash replacement suffered from the greatest damage. Figure 5.8 shows the results from the solutions that showed greater damage, that is 2, 4, and 8% salt. As seen, heavy damage resulted from the freeze-thaw with these salt exposures. Figure 5.9 shows the results from testing in water only, with very little damage across all mixes. The small amount of damage that occurred could likely be the result of the testing process, moving the samples to and from the freezer, 200 times throughout testing. It is unclear why mix 4, with 30% fly ash replacement was weakest in freeze-thaw durability. It was expected that with additional fly ash, the porosity would continue to decrease, and improve the durability. The addition of fly ash may require a longer cure time than the 28 days allowed, which if not reached, would mean exposing uncured concrete to salt and freeze-thaw cycles, the result of which would be severe. The high levels of damage indicate this may have been the case. Figure 5.10 shows the results by mix design for the salt concentrations of 2, 4, 8, and 12% respectively. In Figures 5.9 and 5.10, in each concentration, a band of damage is seen; indicating samples underwent similar testing exposure. The difference being that for mix designs with 10 and 20% the trends had shallower slopes, and slower damage developing.

5.5.3 Scanning Electron Microscope

Through the SEM, the individual particles of the cement and fly ash could be viewed. Figure 5.11 shows mixes with cement only, and with cement and 30% fly ash replacement, respectively as examples. Both images were taken at 11,000 times magnification. In Figure 5.11a, the larger particles of cement can be seen in the absence of fly ash. In Figures 5.11b, fly ash particles, which are an order of magnitude smaller than cement, are seen between the larger cement particles. The images show that with a greater distribution of particle sizes with increasing fly ash, a decrease in the pore sizes can indeed be observed. Other studies have also shown that fly ash replacement in cement has shown increase dispersion in the cement paste, leading to a decrease in average pore size (Chindaprasirt et al, 2005). This reduced porosity would decrease the available void space for solution and ice to move through during the freeze-thaw process, which could lead to a decrease in damage to the cement paste. Additionally, with greater particle distribution, the number of contact points and the strength of the cement paste would increase.

5.6 CONCLUSIONS

A series of laboratory tests was conducted to evaluate the replacing of conventional portland cement binder with fly ash, at four varying ratios in previous concrete. The mix designs were tested for freeze-thaw durability, with exposure to salt solutions. The samples were tested using a modified freeze-thaw procedure, to approximate field conditions. Testing included one freeze-thaw cycle per day, with the sample allowed to drain during freezing.

The effects of fly ash replacement on void ratio, hydraulic conductivity, and compressive strength were also evaluated, and found to be within acceptable ranges for pervious concrete. It was hoped that the inclusion of fly ash would reduce the damage caused by freeze-thaw cycles. Fly ash has a smaller particle size than cement, and by blending the two, the dispersion would increase. This increased distribution of particle sizes would create a cement paste with a lower porosity, and permeability. The following specific conclusions could be drawn from this study:

- Pervious concrete with fly ash replacement of 10, 20, and 30% can provide similar compressive strength, hydraulic conductivity, and void ratio to a control mix using just portland cement.
- Mix designs with 10% and 20% fly ash replacement provided greater durability to freeze-thaw damage, including the use of salt solutions. Replacement of 30% fly ash resulted in comparatively poor performance in freeze-thaw testing with salt solutions.
- Water showed little damage across all samples for freeze-thaw testing. The greatest damage was seen for 8, 4, and 2% salt solutions respectively. For the slow freeze-thaw testing, damage was directly related to salt presence.
- Freeze-thaw damage in all samples was seen where the saturation was the greatest, directly relating increased levels of saturation due to salt solutions with damage.

5.7 REFERENCES

ACI Committee 522 (2010). “Report on Pervious Concrete 522R-10,” American Concrete Institute, Report of ACI Committee 522, ACI 522R-10.

Environmental Protection Agency (2000), “Stormwater Phase II, Final Rule” (Revised December 2005), Office of Water (4203).

Tennis, P. D., Leming, M. L., and Akers, D. J. (2004). “Pervious Concrete Pavements,” Portland Cement Association, Skokie, IL, 15 pp.

Leming, M. L., Malcolm, H. R., Tennis, P. D. (2007). “Hydrologic Design of Pervious Concrete.” *Portland Cement Association*, Skokie, IL.

Schuler, T. (1987) “Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,” *Metropolitan Washington Council of Governments*, Washington, D.C.

National Ready Mixed Concrete Association (NRMCA) (2004). “Freeze-Thaw Resistance of Pervious Concrete.” Silver Spring, MD.

McCain, G. N., and Dewoolkar, M. M. (2010), "Porous concrete pavements: mechanical and hydraulic properties", Transportation Research Record, No. 2164, Washington D.C., 66-75.

Wanielista, Marty and Chopra, Manoj. (2007). “Performance Assessment of Portland Cement Pervious Pavement” Stormwater Management Academy, University of Central Florida

Meininger, R. C. (1988). “No-Fines Pervious Concrete for Paving.” Concrete International, Volume 10, No 8. August.

Wang K., Schaefer V.R., Kevern J.T., Suleiman M.T. (2006). “Development of Mix Proportion for Functional and Durable Pervious Concrete,” Proceedings in CD of the 2006 Concrete Technology Forum, Nashville, National Ready Mix Concrete Association.

ASTM C666 (2008), “Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing,” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Schaefer V.R., Wang, K., Suleiman, M.T., White, D.J., and Kevern, J.T. (2006). “Mix Design Development for Pervious Concrete in Cold Weather Climates,” Final Report, Center for Transportation Research and Education, Iowa State University, Ames, IA, 67 pp.

Kevern, J. T., Schaefer, V. R., Wang, K., and Suleiman, M. T. (2008b), “Pervious Concrete Mixture Proportions for Improved Freeze-Thaw Durability,” *Journal of ASTM International,* ASTM International, West Conshohocken, PA, V. 5, No. 2, pp 12

Ghafoori, N., and Dutta, S. (1995). “Laboratory Investigation of Compacted No-Fines Concrete for Paving Materials,” Journal of Materials in Civil Engineering, Volume 7, Number 3, pp 183-191

Kevern J. T., Wang, K., and Schaefer, V. R. (2008a), “Pervious Concrete in Severe Exposures,” *Concrete International*, V. 30, No. 7, pp 43-49.

Yang, Z., Brown, H., and Cheney, A. (2006), “Influence of Moisture Conditions on Freeze and Thaw Durability of Portland Cement Pervious Concrete,” *Concrete Technology Forum: Focus on Pervious Concrete*, Nashville, TN, pp 15.

Yang, Z. (2011), “Freezing-and-Thawing Durability of Pervious Concrete under Simulated Field Conditions,” *ACI Materials Journal*, V. 108, pp 187-195.

Olek, J., and Weiss, W. J. (2003), “Development of Quiet and Durable Porous Portland Cement Concrete Paving Materials,” Final Report SQDH 2003-5 Center for Advanced Cement Based Materials, Purdue.

Litvan, G.G. (1973), “Pore structure and frost susceptibility of building Materials”, Proc. Int. Symp. on Pore Structure and Properties of Materials (Prague), pp 17–30.

Vuorinnen, J. (1970). “On Use of Dilation Factor and Degree of Saturation in Testing Concrete for Frost Resistance,” Nordisk Betong p. 37

Harnik, A. B., Meier, U. and Rosli, A. (1980), “Combined influence of freezing and deicing salt on concrete - Physical aspects,” ASTM STP-691, pp. 474-484.

Darwin, D., Browning, J., Gong, L., and Hughes, S. R. (2007), “Effects of deicers on concrete deterioration”, Structural Engineering and Materials Laboratory, SL Report 07-3, University of Kansas.

Cody, R. D., Cody, A. M., Spry, P. G., and Gan, G.-L. (1996), “Concrete Deterioration by Deicing Salts: An Experimental Study,” *Proceedings*, Semisequicentennial Transportation Conference, Iowa State University, Ames, Iowa.

Lee, H., Cody, A. M., Cody, R. D., and Spry, P. G. (2000), “Effects of Various Deicing Chemicals on Pavement Concrete Deterioration,” *Proceedings*, Mid-Continent Transportation Symposium, Center for Transportation Research and Education, Iowa State University, Ames, Iowa, pp. 151-155.

Cloutier, J., and Newbury, G. (2009), “Salt Brine, Salt Brine Blends and Application Technologies During the 2008-2009 Winter Maintenance Season” VT DOT, Report 2009-9

Verbeck, G. and Klieger, P. (1957), “Studies of ‘Salt’ Scaling,” *Research Department Bulletin* 83, Portland Cement Association, Chicago, Illinois, 13 pp.

Marchand, J., Pigeon, M., Bager, D., and Talbot, C., (1999) “Influence of Chloride Solution Concentration on Deicer Salt Scaling Deterioration of Concrete,” ACI Materials Journal, Vol. 96, No. 4, pp. 429-435.

Chan, C., Hover, K., Folliard, K., Hance, R., Trejo, D. (2007), “Durability of Segmental Retaining Wall Blocks: Final Report,” Research, Development and Technology, Federal Highway Administration, Publication No. FHWA-HRT-07-021.

Pigeon, M. and Pleau, R. (1995), “Durability of Concrete in Cold Climates,” E&FN SPON (an imprint of Chapman & Hall), London, UK, first edition.

Chindaprasirt. P., Jaturapitakkul C., Sinsiri T. (2005). “Effect of fly ash fineness on compressive strength and pore size of blended cement paste.” Cement Concrete Composites 27 (4) (2005) 425–428.

ASTM C618 (2008). “Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Portland Cement Association (PCA) (2002). “Designs and Control of Concrete Mixtures 14th Edition” Portland Cement Association, Skokie, IL.

Helmuth R (1987). “Fly ash in cement and concrete.” Portland Cement Association; 1st ed. Illinois; 1987.

Chindaprasirt P, Homwuttiwong S, Sirivivatnanon V. (2004). “Influence of fly ash fineness on strength, drying shrinkage and sulfate resistance of blended cement mortar.” Cement Concrete Research 2004; 34:1087–92.

ASTM C192 (2008), “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory,” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Park, S., and M. Tia. (2004). “An experimental study on the water-purification properties of porous concrete.” Cement and Concrete Research 34,177-184

ASTM C39 (2008). “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C1231 (2008). “Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Montes, F. and Haselbach, L.M. (2006). “Measuring Hydraulic Conductivity in Pervious Concrete.” Environmental Engineering Science. Nov 2006, Vol. 23, No. 6, pp. 960-969.

Bentz, D. P.; Sant, G.; and Weiss, W. J. (2008). "Early-Age Properties of Cement-Based Materials—I: Influence of Cement Fineness," Journal of Materials in Civil Engineering, ASCE, V. 20, No. 7, July 2008, pp. 502-508.

Table 0.1 - Pervious concrete mix designs used in this study

Mix Design	Aggregate (kg)	Water (kg)	Cement (kg)	Fly Ash (kg)	Admixtures	Fly Ash (%)	W/C	A/C
1	84.55	6.67	19.05	0	AEA:3.95 ml HRWR:24.89 ml VMA:60.18 ml STAB:60.18 ml	0	0.35	4.4
2	84.55	6.67	17.15	1.9		10	0.35	4.4
3	84.55	6.67	15.24	3.81		20	0.35	4.4
4	84.55	6.67	13.34	5.71		30	0.35	4.4

Table 0.2 –Summary of average engineering properties

Mix Design	Fly Ash (%)	Density (kg/m^3)	Void Ratio	Hydraulic Conductivity (cm/s)	Compressive Strength (MPa)
1	0	1988.3	0.352	1.37	11.03
2	10	1969.1	0.348	1.18	10.76
3	20	1950.0	0.351	1.23	9.45
4	30	1954.6	0.354	1.19	9.02

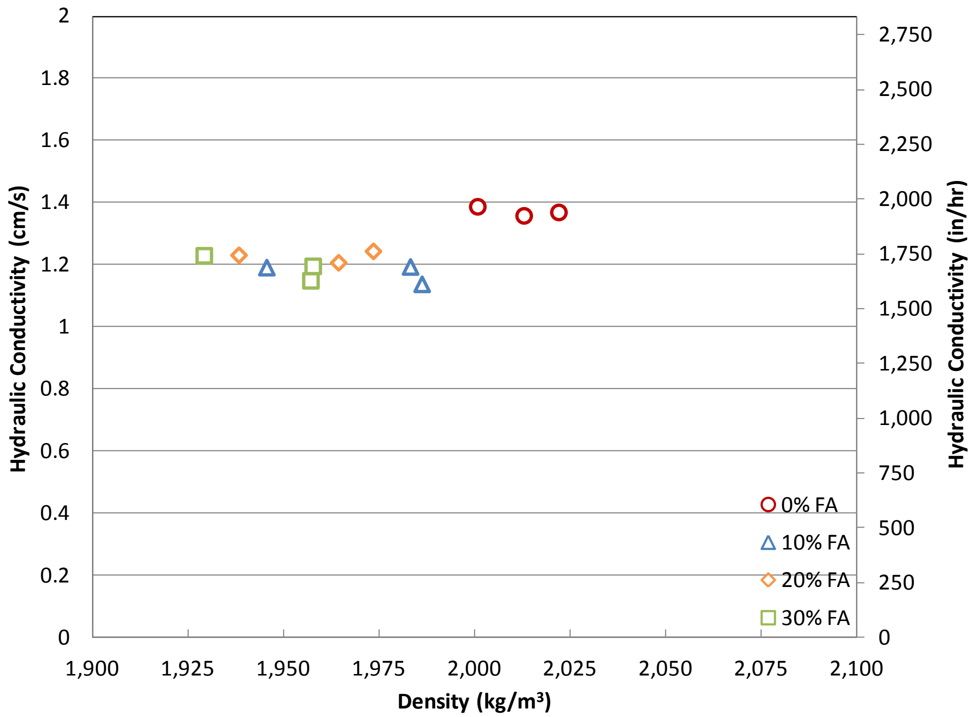


Figure 0.1 - Hydraulic Conductivity: Per Mix Design

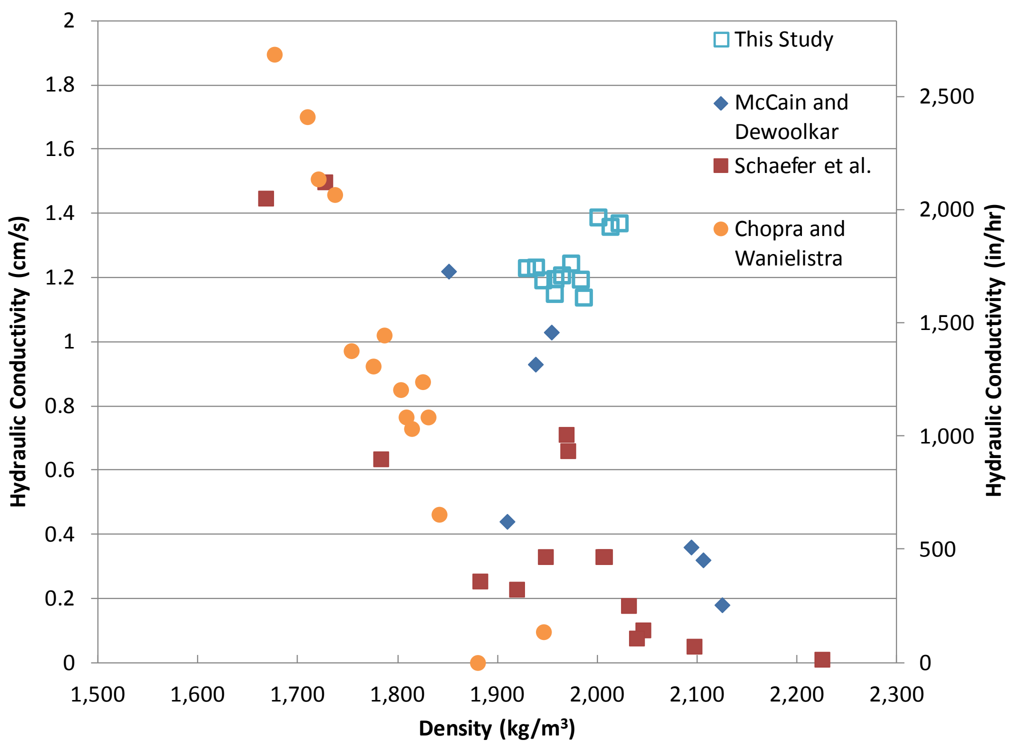


Figure 0.2 – Hydraulic Conductivity Comparison

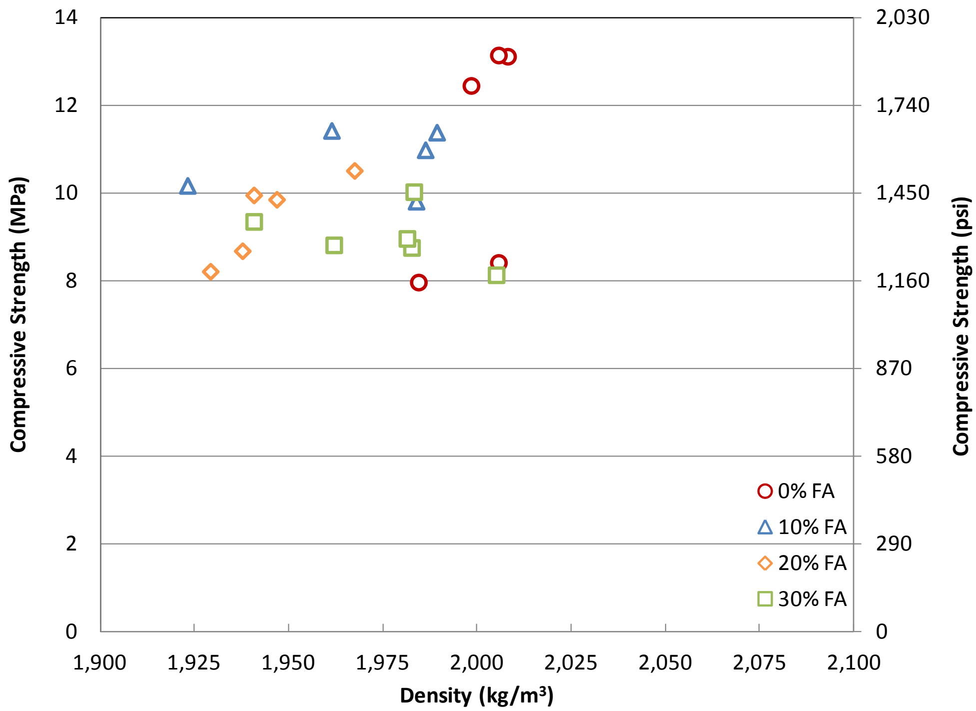


Figure 0.3 - Compressive Strength: Per Mix Design

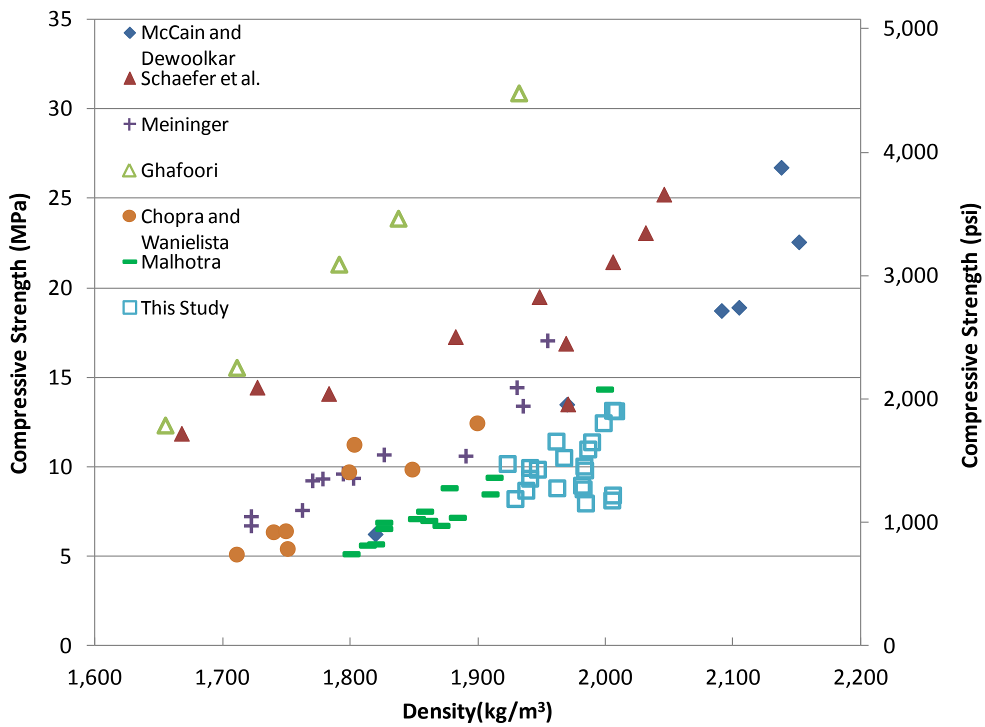


Figure 0.4 - Compressive Strength Comparison

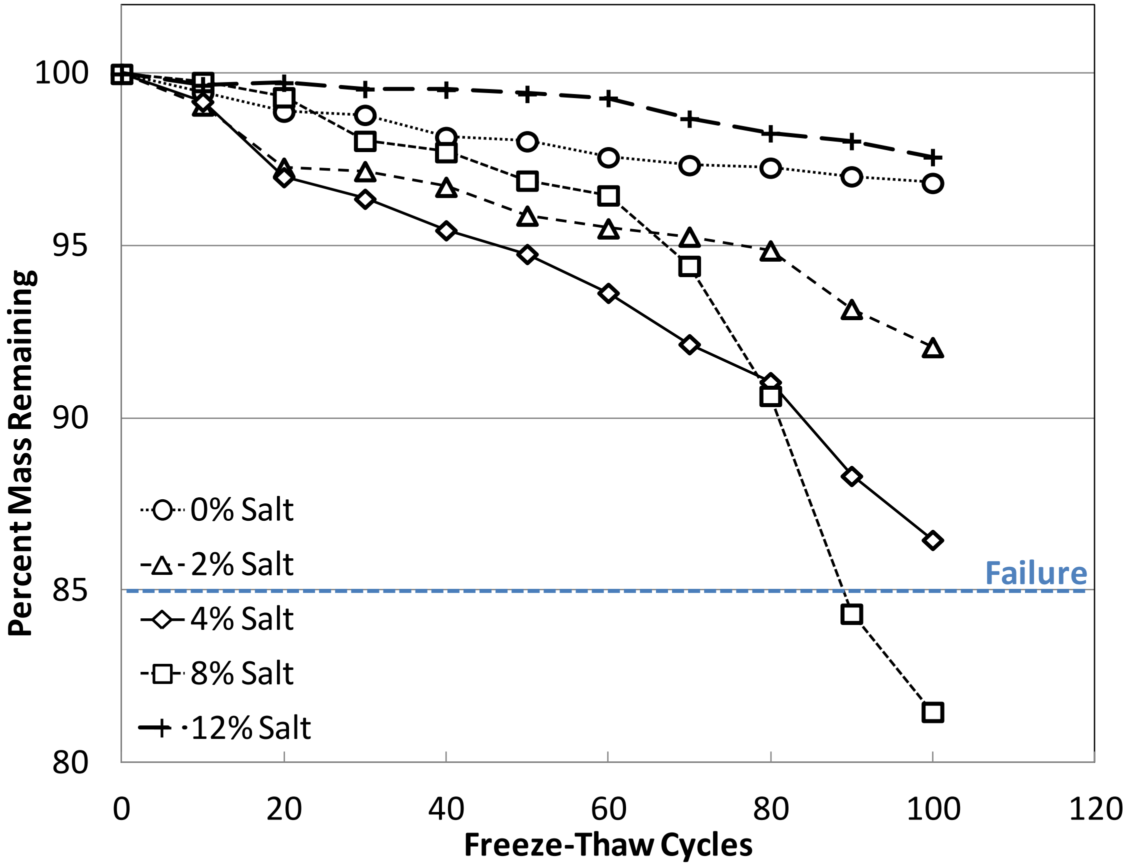


Figure 0.5 - Freeze-Thaw: All Mix Designs per Salt Concentration

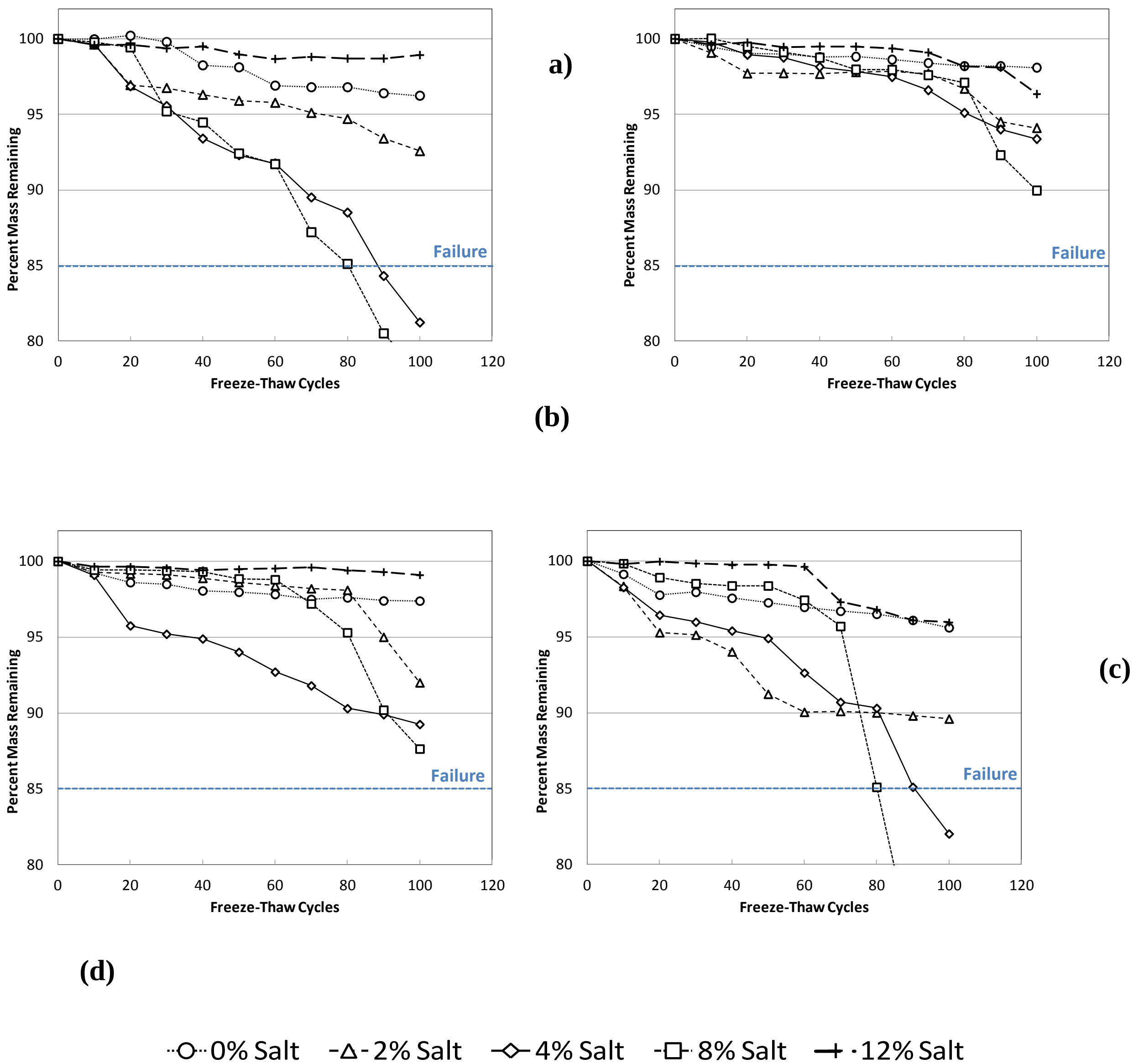


Figure 0.6 - Freeze-Thaw: Mix Designs, per Salt Concentration

(a) 0% Fly Ash, (b) 10% Fly Ash, (c) 20% Fly Ash, (d) 30% Fly Ash

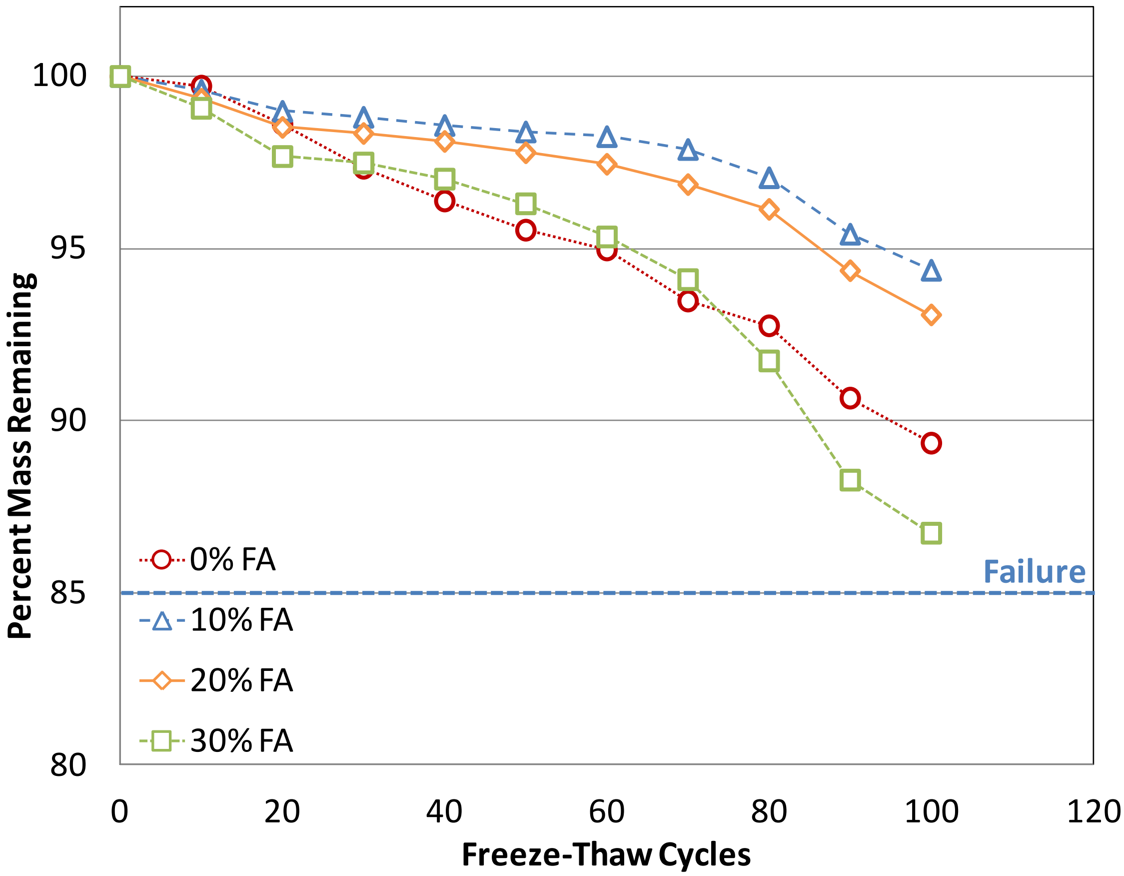


Figure 0.7 - Freeze-Thaw: All Solutions, per Mix Design

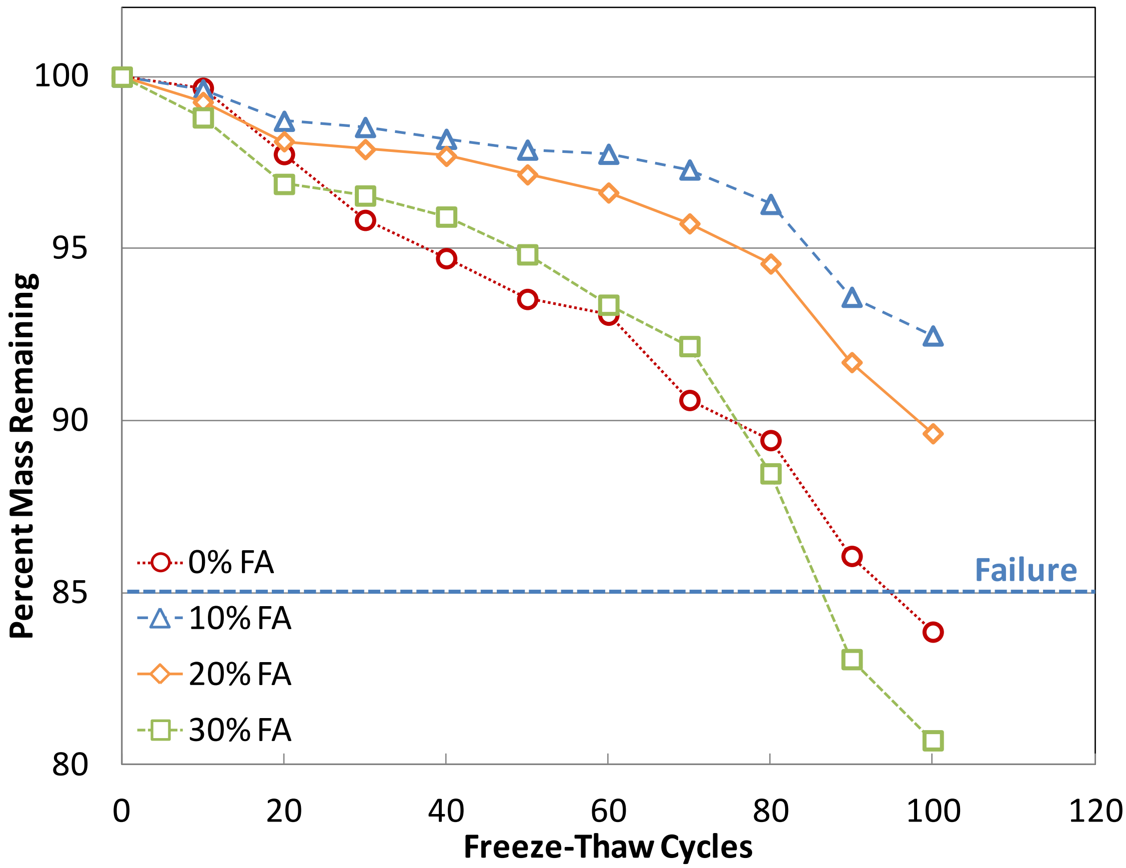


Figure 0.8 - Freeze-Thaw: 2, 4, and 8% Salt Solutions, per Mix Design

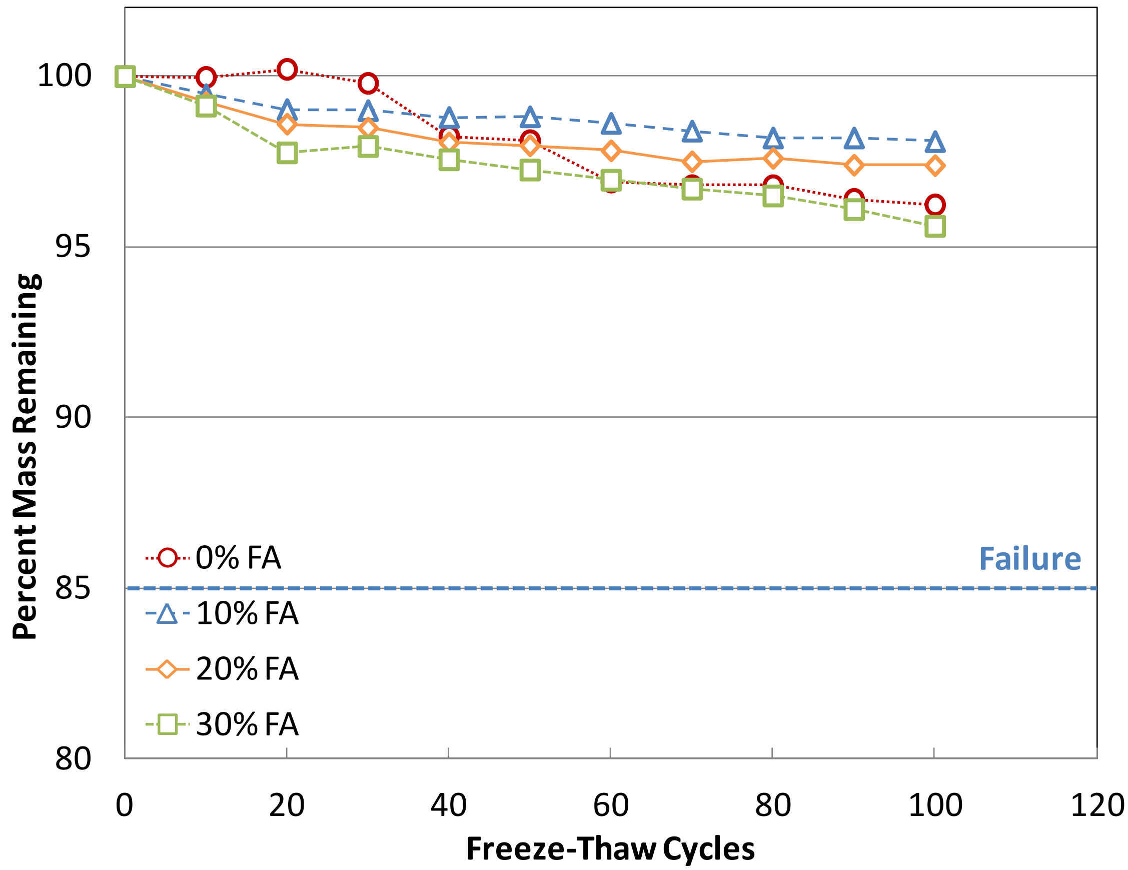
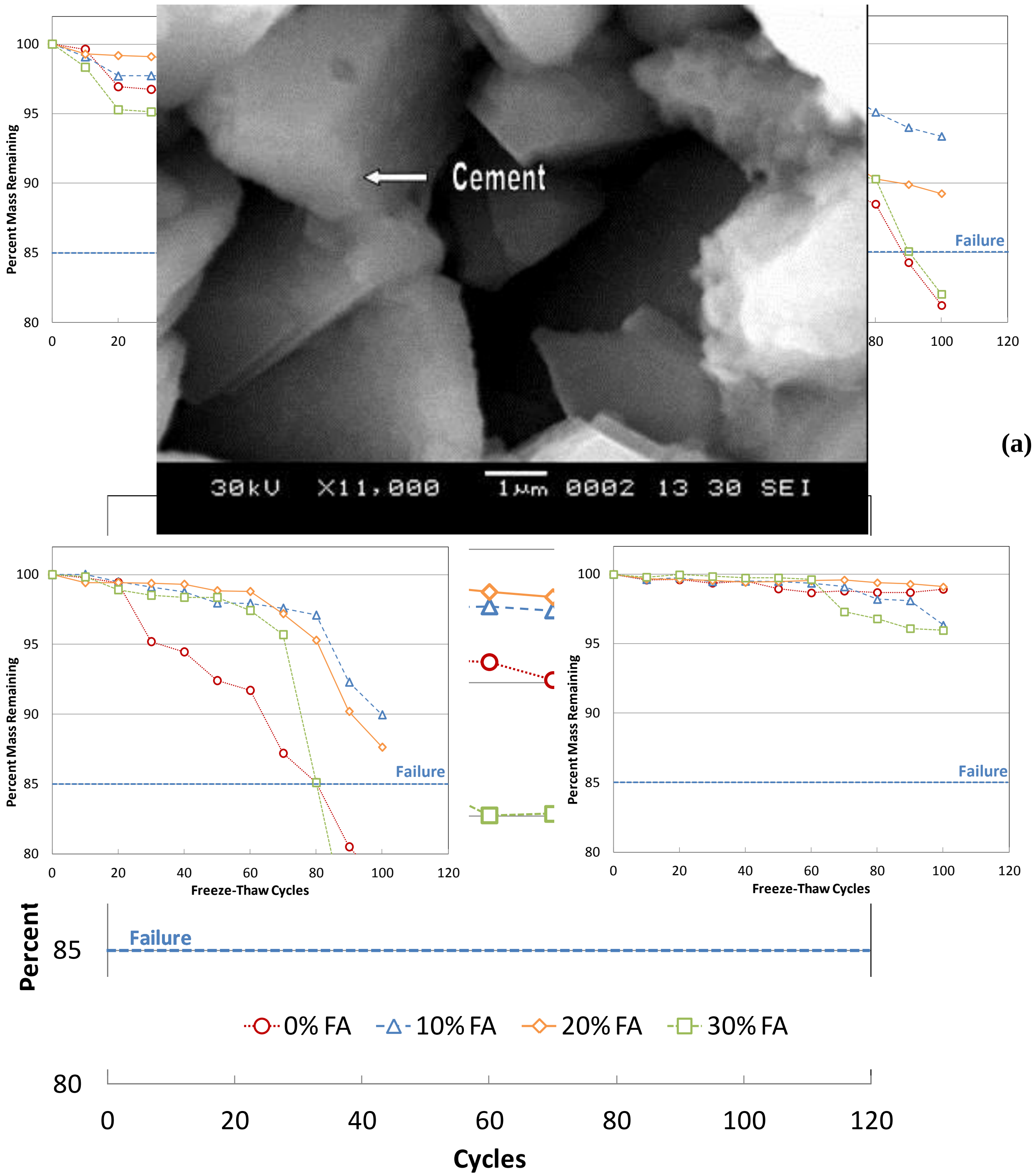
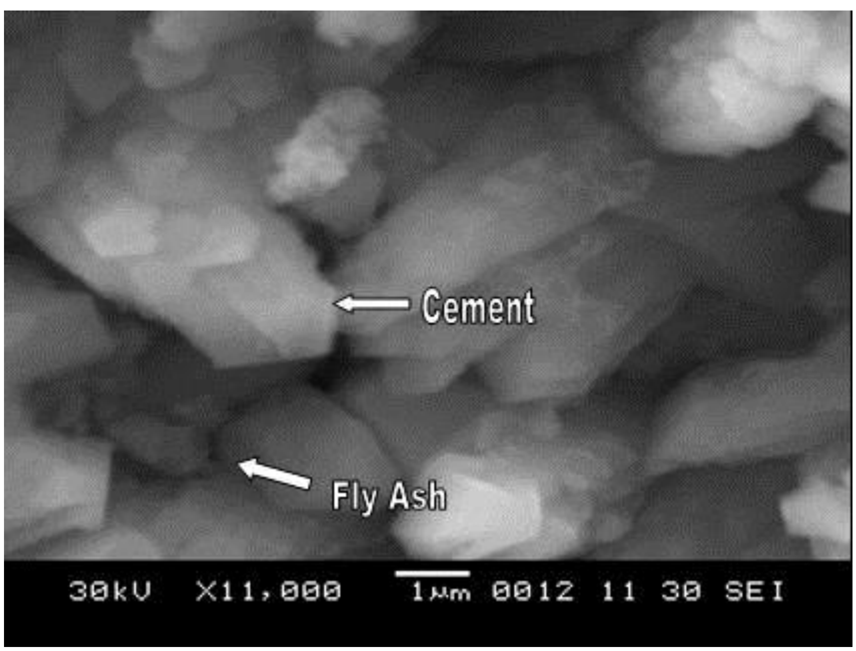


Figure 0.9 - Freeze-Thaw: 0% Salt Solution (Water), per Mix Design


Cement
30kV X11,000 1µm 0002 13 30 SEI
(a)
Percent Mass Remaining
Freeze-Thaw Cycles
Failure
Percent
Cycles
0% FA
10% FA
20% FA
30% FA

(a)



(b)

Figure 0.11 - Scanning Electron Microscope Observations:

(a) Pervious Concrete with Cement only; (b) Pervious Concrete with Fly Ash

CHAPTER 6

CONCLUSIONS AND FUTURE RECOMMENDATIONS

6.1 CONCLUSIONS

This chapter summarizes the conclusions drawn from this study that investigated laboratory and field testing and field behavior of pervious concrete. Recommendations for future research are also made.

Specimens capped with rubber pads showed a better relation between compressive strength and unit weight compared to sulfur capping for specimens with a H:D ratios of 2:1 and 1.5:1. In addition, specimens capped with rubber pads were found to have lower standard deviations values for compressive strength measurements compared to sulfur capped specimens for most of the mix designs investigated in this study. Based on these observations rubber capping methods are recommended over sulfur capping for compressive strength testing of pervious concrete with H:D ratios of 2:1 and 1.5:1.

Specimens with H:D ratios less than 2:1 were found to have compressive strength measurements 10% greater than those of the recommended ratio of 2:1. The relation between measured compressive strength and unit weight was found to differ based upon H:D ratio. Reliable correlations between compressive strength measurements and unit weight values were found for specimens with H:D ratios of 2:1 and 1.5:1. Specimens with H:D ratios of 1:1 exhibited relatively inconsistent relationships between compressive strength measurements and unit weight. Measurements of compressive strength of pervious concrete is recommended to be performed with specimens having a H:D ratio of 2:1; if this ratio cannot be achieved specimens with H:D ratios less than the recommendation can be used and results reduced by 10% to approximate the compressive strength of specimens with a 2:1 H:D ratio. Specimens with a H:D ratio of 1:1 should be avoided if possible.

The infiltration testing methods evaluated in this study were found to have linear correlations to one another. The infiltration rates from single ring, double ring and falling head infiltrometers correlated as 1: 0.85: 4.7 for field sites. The double ring infiltrometer was found to provide meaningful measurements only at locations where infiltration rates were less than about 150 in/hr (0.10 cm/sec). This limitation was due to an inability to maintain a constant level of water between the inner and outer rings at higher infiltration values. Hydraulic conductivity correlated linearly with the infiltration rates from the single ring and falling head infiltrometers as 1: 1.8: 9.0 in laboratory testing. These correlations can be used to estimate hydraulic conductivity of pervious concrete from field measurements of infiltration rates. All testing was performed on 6” thick pervious concrete; however, it is anticipated that these relations will be generally valid for sections with different thicknesses.

Average infiltration rates at the College Street and Heritage facilities decreased by 59% and 26% respectively, over the monitoring period. Gradual reductions to infiltration rate were observed at both sites, indicating that the clogging process occurred consistently irrespective of the season. Differences between sites inside and outside of the wheel track of vehicles indicated that the presence of vehicle travel can increase clogging. Clogging was found to be more severe in locations where trucks regularly traveled compared to other locations. Reductions to permeability ranged from 20% to 40% in previous laboratory studies that simulated clogging.

This suggests that the laboratory studies predicted the clogging of pervious concrete in the field reasonably well.

Street sweeping and vacuum truck cleaning performed as facility wide cleaning operations increased average infiltration rates by 21% and 30% respectively. Spot cleaning methods showed increases to infiltration rate of 85% after pressure washing, 10% after vacuuming and 100% after pressure washing followed by vacuuming. Both pressure washing and combining pressure washing and vacuuming were found to increase infiltration rates in areas of severe clogging.

The long-term monitoring and cleaning of PCP results indicated that PCP facilities should be periodically cleaned using facility wide cleaning operations twice a year, preferably once in the late fall and once in the spring. Cleaning should be performed with a vacuum truck when possible; however, a street sweeper can be used in cases where vacuum truck cleaning is not possible. Cleaning operations should be conducted immediately after construction to prevent the formation of severely clogged locations. If severely clogged locations are present infiltration rates can be restored by pressure washing or a combination of pressure washing and vacuum cleaning.

Pervious concrete with fly ash replacement of 10, 20, and 30% can provide similar engineering properties to a control mix using just Portland cement. Mix designs with 10% and 20% fly ash replacement provided greater resistance to freeze-thaw damage, including the use of salt solutions. Replacement of 30% fly ash resulted in comparatively poor performance in freeze-thaw testing with salt solutions. Water showed little damage across all samples for freeze-thaw testing. The greatest damage was seen for 8, 4, and 2% salt solutions respectively. For the slow freeze-thaw testing, damage was directly related to salt presence. Freeze-thaw damage in all samples was seen where the saturation was the greatest, directly relating increased levels of saturation due to salt solutions with damage.

6.2 CORRELATION TO FIELD OBSERVATIONS

Results from this work can be used to support some of the observations seen in pervious concrete throughout Vermont. The recovery of infiltration capacity when cleaning methods were used, suggest that sites like the College St. and Heritage would have had similar performance measures if the same maintenance was followed. Clogging of the pore can be prevented with regular seasonal cleaning, but if sites are allowed to be clogged for extended periods of time, the performance may never be recovered.

The laboratory results from the freeze-thaw testing follow what was seen in the field in comparing the Heritage site where no salt is used, and the Randolph site where salt was used. The Randolph site has seen widespread surface scaling, as well as internal degradation of the pervious concrete. A section of the Randolph site which is not salted has shown much better performance, and may be the result of not being exposed to salt during freeze-thaw. The Heritage site has seen very little damage, with isolated locations where salt was transported onto the site by vehicle traffic. Areas of clogging and increased saturation also showed increases in damage, for all sites with and without salt exposure. Sites that did not use salt, and were periodically maintained, as Heritage was, outperform those without maintenance, College St, and those that use salt and are not maintained, as with Randolph. Additionally, the Heritage lot, which has some replacement of Portland cement with fly ash, shows better durability than other sites where only conventional cement was used.

6.3 RECOMMENDATIONS

Based on the study presented here, the following recommendations are made for future instillations and research of pervious concrete.

- Maintenance should be done twice a year to prevent clogging, and ensure the continued performance of the pervious surface. Ideally vacuuming would be used.
- Monitoring should continue on existing pervious concrete pavements, to observe structural and hydraulic performance as time progresses.
- The use of fly ash at 10 and 20% can be incorporated to improve freeze-thaw durability and salt resistance in pervious concrete.
- The use of deicing salts results in damage during freeze-thaw, its application should be avoided, delayed, or at a minimum limited.
- Mix design development including fine aggregate and cementitious alternative to reduce paste permeability, should be investigated to improve freeze-thaw resistance.
- The relation between compaction effort and curing conditions to freeze-thaw resistance and strength in a field representative environment should be investigated.

COMPREHENSIVE BIBLIOGRAPHY

ACI Committee 522 (2010). “Report on Pervious Concrete 522R-10,” American Concrete Institute, Farmington Hills, Mich., 2010.

ASTM C39 (2010). “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C192 (2010). “Standard Practice for Making and Curing Concrete Test Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C617 (2010). “Standard Practice for Capping Cylindrical Concrete Specimens.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C618 (2010). “Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C666 (2010), “Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing,” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C1231 (2010). “Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

ASTM C1701 (2010). “Standard Test Method for Infiltration Rate of In Place Pervious Concrete.” Annual Book of ASTM Standards 4.02. West Conshohocken, PA: ASTM International.

Bean, E. Z., Hunt, W. F., and Bidelspach, D. A. (2007). “Field Survey of Permeable Pavement Surface Infiltration Rates.” J. of Irrigation and Drainage Engineering. Vol. 133, No. 3, pp. 249-255.

Berry B., Anderson I., Suozzo M., Dewoolkar M.M. (2012), "Properties of Pervious Concrete Incorporating Recycled Concrete Aggregate", Accepted as part of the Transportation Research Board Annual Meeting. January, 2012. Washington D.C.

Bentz, D. P.; Sant, G.; and Weiss, W. J. (2008). “Early-Age Properties of Cement-Based Materials—I: Influence of Cement Fineness,” Journal of Materials in Civil Engineering, ASCE, V. 20, No. 7, July 2008, pp. 502-508.

Chan, C., Hover, K., Folliard, K., Hance, R., Trejo, D. (2007), "Durability of Segmental Retaining Wall Blocks: Final Report," Research, Development and Technology, Federal Highway Administration, Publication No. FHWA-HRT-07-021.

Chindaprasirt P, Homwuttiwong S, Sirivivatnanon V. (2004). "Influence of fly ash fineness on strength, drying shrinkage and sulfate resistance of blended cement mortar." Cement Concrete Research 2004; 34:1087–92.

Chindaprasirt P., Hatanaka S., Chareerat T., Mishima N. and Yuasa Y.(2008). "Cement Paste Characteristics and Porous Concrete Properties." Construction and Building Materials, Volume 22, Issue 5, May 2008, pp. 894-901.

Chindaprasirt. P., Jaturapitakkul C., Sinsiri T. (2005). "Effect of fly ash fineness on compressive strength and pore size of blended cement paste." Cement Concrete Composites 27 (4) (2005) 425–428.

Chopra, M., Kakuturu, S., Ballock, C. Spence, J. (2010). "Effect of Rejuvenation Methods on the Infiltration Rates of Pervious Concrete Pavements" Journal of Hydraulic Engineering, Vol. 15, No. 6, June 2010.

Chopra, M. and Wanielista, M. (2007). "Performance Assessment of Portland Cement Pervious Pavement: Compressive Strength of Pervious Concrete Pavements." Stormwater Management Academy, University of Central Florida.

Cloutier, J., and Newbury, G. (2009), "Salt Brine, Salt Brine Blends and Application Technologies During the 2008-2009 Winter Maintenance Season" VT DOT, Report 2009-9

Cody, R. D., Cody, A. M., Spry, P. G., and Gan, G.-L. (1996), "Concrete Deterioration by Deicing Salts: An Experimental Study," *Proceedings*, Semisequicentennial Transportation Conference, Iowa State University, Ames, Iowa.

Darwin, D., Browning, J., Gong, L., and Hughes, S. R. (2007), "Effects of deicers on concrete deterioration", Structural Engineering and Materials Laboratory, SL Report 07-3, University of Kansas.

Deo, O., Sumanasooriya, M., and Neithalath, N. (2010)."Permeability Reduction in Pervious Concretes due to Clogging: Experiments and Modeling" Journal of Materials in Civil Engineering, Vol. 22, No. 7, July 1, 2010.

Environmental Protection Agency (2000), "Stormwater Phase II, Final Rule" (Revised December 2005), Office of Water (4203).

Ferguson, Bruce K. (2005), Porous Pavements, Boca Raton, FL: CRC Press.

Ghafoori, N., and Dutta, S. (1995). "Laboratory Investigation of Compacted No-Fines Concrete for Paving Materials." Journal of Materials in Civil Engineering, Vol. 7, No. 3, August 1995, pp.

183-191.

Guthrie, W. S., DeMille, C. B., Eggett, D. L. (2010). “Effects of Soil Clogging and Water Saturation on Freeze–Thaw Durability of Pervious Concrete” Transportation Research Record: Journal of the Transportation Research Board No. 2164, Washington, DC, pp. 88-97.

Harnik, A. B., Meier, U. and Rosli, A. (1980), “Combined influence of freezing and deicing salt on concrete - Physical aspects,” ASTM STP-691, pp. 474-484.

Head W.J., Iskander, W.H., Folden, J.D. (1999). “Assessment of Steel End Caps with Rubber Inserts as Capping Media for Concrete Cylinders.” Final Report WVDOH RP# 136. West Virginia University.

Helmuth R (1987). “Fly ash in cement and concrete.” Portland Cement Association; 1st ed. Illinois; 1987.

Henderson, V., Tighe, S.L., and Norris, J. (2009). “Pervious Concrete Pavement: Integrated Laboratory and Field Study.” Transportation Research Record: Journal of the Transportation Research Board. No. 2113, Washington, DC, pp. 13-21.

Joung, Y., and Grasley, Z. C. (2008). “Evaluation and Optimization of Durable Pervious Concrete for Use in Urban Areas.” *Technical Report for the Texas Transportation Institute, Texas A&M University.* February. College Station, TX

Kevern, J. T., Schaefer, V. R., Wang, K., and Suleiman, M. T. (2008b), “Pervious Concrete Mixture Proportions for Improved Freeze-Thaw Durability,” *Journal of ASTM International*, ASTM International, West Conshohocken, PA, V. 5, No. 2, pp 12

Kevern J. T., Wang, K., and Schaefer, V. R. (2008a), “Pervious Concrete in Severe Exposures,” *Concrete International*, V. 30, No. 7, pp 43-49.

Klieger P. and Lamond J. F. (1994). “Significance of Tests and Properties of Concrete and Concrete-Making Materials” American Standard Testing Methods, Fredericksburg, VA.

Lee, H., Cody, A. M., Cody, R. D., and Spry, P. G. (2000), “Effects of Various Deicing Chemicals on Pavement Concrete Deterioration,” *Proceedings,* Mid-Continent Transportation Symposium, Center for Transportation Research and Education, Iowa State University, Ames, Iowa, pp. 151-155.

Leming, M. L., Malcolm, H. R., Tennis, P. D. (2007). “Hydrologic Design of Pervious Concrete.” *Portland Cement Association*, Skokie, IL.

Litvan, G.G. (1973), “Pore structure and frost susceptibility of building Materials”, Proc. Int. Symp. on Pore Structure and Properties of Materials (Prague), pp 17–30.

Marchand, J., Pigeon, M., Bager, D., and Talbot, C., (1999) “Influence of Chloride Solution Concentration on Deicer Salt Scaling Deterioration of Concrete,” ACI Materials Journal, Vol. 96, No. 4, pp. 429-435.

McCain, G. N., and Dewoolkar, M. M. (2010), "Porous concrete pavements: mechanical and hydraulic properties", Transportation Research Record, No. 2164,Washington D.C., 66-75.

Meininger, R.C., “No-Fines Pervious Concrete for Paving,” Concrete International, Vol. 10, No. 8, August 1988, pp. 20-27.

Montes, F. and Haselbach, L.M. (2006). “Measuring Hydraulic Conductivity in Pervious Concrete.” Environmental Engineering Science. Nov 2006, Vol. 23, No. 6, pp. 960-969.

National Ready Mixed Concrete Association (NRMCA) (2004). “Freeze-Thaw Resistance of Pervious Concrete.” Silver Spring, MD.

Olek, J., and Weiss, W. J. (2003), “Development of Quiet and Durable Porous Portland Cement Concrete Paving Materials,” Final Report SQDH 2003-5 Center for Advanced Cement Based Materials, Purdue.

Park, S., and M. Tia. (2004). “An experimental study on the water-purification properties of porous concrete.” Cement and Concrete Research 34,177-184

Pigeon, M. and Pleau, R. (1995), “Durability of Concrete in Cold Climates,” E&FN SPON (an imprint of Chapman & Hall), London, UK, first edition.

Portland Cement Association (PCA) (2002). “Designs and Control of Concrete Mixtures 14th Edition” Portland Cement Association, Skokie, IL.

Portland Cement Association (PCA) (2003). “Building Green with Concrete: Points for Concrete in LEED 2.1, IS312.” Portland Cement Association, Skokie, IL.

Rizvi, R., Tighe, S., Henderson, V., and Norris, J. (2009). “Laboratory sample preparation techniques for Pervious Concrete.” TRB Annual Meeting. Report No. 09-1962.

Sansalone, J., Kuang, X., and Ranieri, V. (2008). “Permeable Pavement as a Hydraulic and Filtration Interface for Urban Drainage.” Journal of Irrigation and Drainage Engineering Volume: 134. Issue: 5. Pages: pp. 666-674.

Schaefer, V., Wang, K., Suleimman, M. and Kevern, J. (2006). “Mix Design Development for Pervious Concrete in Cold Weather Climates.” Final Report, Civil Engineering, Iowa State University.

Schuler, T. (1987) “Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,” *Metropolitan Washington Council of Governments*, Washington, D.C.

Suozzo, M., and Dewoolkar, M.M. (2011). “Evaluation of Strength and Hydraulic Testing Methods of Pervious Concrete” Submitted to ACI Materials Journal November 2011 (In Review).

Tennis, P. D., Leming, M. L., and Kiefer, C. J. (2004). “Pervious Concrete Pavements.” Portland Cement Association, Skokie, IL.

Verbeck, G. and Klieger, P. (1957), “Studies of ‘Salt’ Scaling,” *Research Department Bulletin* 83, Portland Cement Association, Chicago, Illinois, 13 pp.

Vuorinnen, J. (1970). “On Use of Dilation Factor and Degree of Saturation in Testing Concrete for Frost Resistance,” Nordisk Betong p. 37

Wang K., Schaefer V.R., Kevern J.T., Suleiman M.T. (2006). “Development of Mix Proportion for Functional and Durable Pervious Concrete,” Proceedings in CD of the 2006 Concrete Technology Forum, Nashville, National Ready Mix Concrete Association.

Wanielista, Marty and Chopra, Manoj. (2007). “Performance Assessment of Portland Cement Pervious Pavement” Stormwater Management Academy, University of Central Florida

Yang, Z., Brown, H., and Cheney, A. (2006), “Influence of Moisture Conditions on Freeze and Thaw Durability of Portland Cement Pervious Concrete,” *Concrete Technology Forum: Focus on Pervious Concrete*, Nashville, TN, pp 15.

Yang, Z. (2011), “Freezing-and-Thawing Durability of Pervious Concrete under Simulated Field Conditions,” *ACI Materials Journal*, V. 108, pp 187-195.